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June, 1959

Papers

Journal of the
HYDRAULICS DIVISION
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GROUND-WATER PROBLEMS IN NEW YORK AND NEW ENGLAND[a]

Joseph E. Upson[1]

ABSTRACT

Some ground-water problems in New York and New England involve only a determination of the occurrence and quality of water in geologic formations, and estimates of the productivity of wells. Other problems, such as sea-water encroachment in Long Island, require advanced knowledge of the hydraulics of ground water movement.

Increasingly, the problems involve the interrelationship between ground water and surface water, as in: (1) estimates of how changes in stream regimen affect ground-water conditions; (2) relative practicability of developing surface water and ground water for particular needs; or (3) measurement of surface discharge to indicate amounts of ground water available.

An example of the first type of problem is that in the Ipswich River drainage basin in Massachusetts. Here, investigations are related in part to indicating the possible effect on nearby municipal wells of artificially lowering the river channel.

An example of the second type is that in the Blackstone River valley, Rhode Island. The problem is to determine whether it is practicable to obtain additional water for the city of Woonsocket, and for industries downstream, from ground water or from upstream surface reservoirs.

An example of the third type is that in the Pawcatuck River drainage basin, Rhode Island, where a quantitative estimate of ground water available to supply a proposed industrial park is needed. Location of productive water-bearing strata can be based on ground-water studies alone, but estimation of the total amount of water available can best be done by measuring river outflow and estimating evapotranspiration losses from swamps and lakes.

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a. Prepared in cooperation with Massachusetts Department of Public Works; Rhode Island Development Council; Connecticut State Water Commission; New York State Water Power and Control Commission.

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INTRODUCTION

This paper discusses the changing nature of ground-water problems in New York and New England, in the light of modern uses of water and needs for information, and is intended particularly to show that quantitative and other specific ground-water problems in most of the area necessarily involve surface water also.

Surface water is defined as water that occurs in streams, lakes, and swamps on the surface of the earth. Ground water is defined as water that occurs in the zone of saturation in the rocks and deposits beneath the surface of the earth. Both surface water and ground water are in that part of the hydrologic cycle which intervenes between rainfall, on the one hand, and the water in and evaporating from the oceans, on the other. Locally and occasionally, ground water is considered to be a separate resource, independent of surface water. One of the purposes of this paper is to show that this is not necessarily the case.

Of the water that falls on the land surface as rain, some moves directly into streams, and some percolates directly underground. Of the water that moves underground, what is not picked up by plants or evaporated directly back into the atmosphere discharges as springs or into lakes and streams. Conversely, under proper conditions water in streams can percolate underground and become ground water. Thus there is, in a broad sense, an interchange between surface water and ground water; and both are parts of the same resource.

Change in Types of Ground-Water Problems

In early years people simply needed to know where to dig, or drill, a well for a single household or farm supply. Later, and even now in many parts of New York and New England, it was necessary to know only where to drill a well in order to find substantial supplies—in other words, merely to locate sizable ground-water reservoirs and to estimate the productivity of wells. The amount of water needed was a small part of the total available, and there was no need to estimate total quantities available in a whole area. Nowadays, however, tremendous quantities of ground water are used. The reasons for this large use include the relative constancy of the temperature of ground water, its uniform and often good quality, its availability without transport through long pipelines, and its relative cheapness of development. Of course, the increased demand for ground water also is a result of new uses for water, as in air conditioning, as well as simply for more water for the same old purposes. As the new uses and increased demands approach the total available supply in any one area or location, it becomes important to know the amount of that supply fairly accurately.

The accompanying table gives approximate figures on the withdrawals of fresh ground water in New York and New England in 1950 and 1955. The data are presented primarily to demonstrate that large quantities of ground water are being used in modern times. Examination of the table shows that, in some States and for some purposes, the amount of ground water withdrawn in 1955 actually was less than in 1950. These decreases result from (1) a change, in some areas, from ground water to surface water as a source of supply, (2) cessation of operations entirely by some users of ground water, and (3)

Estimated withdrawals of fresh ground water, New York and New England, 1950 and 1955

STATE	RURAL Domestic and stock (mgd)		MUNICIPAL (mgd)		INDUSTRIAL From private sources (mgd)		IRRIGATION (Acre-ft/yr)	
	1950	1955	1950	1955	1950	1955	1950	1955
New York	160	110	250	260	280	250	30,000	25,000
Connecticut	15	8	9	10	40	51	1,000	1,500
Maine	9	5	8	6	15	10	-	210
Massachusetts	40	7	75	82	100	65	3,000	200
New Hampshire	15	3	7	9	10	4	-	20
Rhode Island	7	1	15	10	15	11	-	50
Vermont	15	8	10	11	5	12	-	-
Total	261	142	374	388	465	403	34,000	26,980

Figures for 1950 from MacKichan, 1951, U. S. Geol. Survey Circular 115.

Figures for 1955 from MacKichan, 1957, U. S. Geol. Survey Circular 398.

appreciably more rainfall in southern New England and New York in the summer of 1955 than in the summer of 1950.

The increase in withdrawals for municipal use in New York and most of the New England States is probably in accord with the general nationwide increase in water use. The withdrawals of ground water are large in comparison with those at the turn of the century.

Examples of Current Problems

In addition to large demands for water, a more intensive use of land, as in the conversion from agricultural use to urban and suburban use, also results in the necessity for studying conditions of ground-water occurrence. For example, certain towns in Connecticut that anticipate or are now experiencing marked urban and suburban development feel the need of zoning regulations to control the size and number of home lots. This is on the basis that each home will have its own supply of water and will dispose of its own wastes. The problems are (1) how many domestic wells will the ground water in these areas sustain? and (2) what are the possibilities of contamination of the water of any or all of them? The solutions of these problems depend in part on understanding the detailed occurrence and amount of available ground water in each area.

Another example of the need for detailed information on the occurrence and availability of ground water is found on Long Island. Here the main source of water in most of the island is ground water. Shortages of supply have been experienced in the western part of the island, and population and industrial growth, accomplished in Nassau County and expected in Suffolk County, have raised and will raise the withdrawal of ground water to magnitudes approaching the total available supply. On Long Island this is largely a ground-water problem and could be solved by examination and study of ground-water conditions alone. These studies involve complex hydrologic and hydraulic problems. However, even on Long Island most of the ground-water discharge is into streams, so the surface-water discharge is in part a measure of the total ground water available.

This paper particularly discusses three problems in New England that were initially ground-water problems, but that involve surface water directly either because stream flow is inherent in the problem, or because measurements of surface-water discharge constitute a good way of obtaining the quantitative ground-water results. These problems are in the drainage basins of (1) the Ipswich River, Mass., (2) the Blackstone River, Mass. and R. I., and (3) the Pawcatuck River, R. I. (See Fig. 1.)

The geologic conditions elsewhere in New England and in most of New York State are such that problems similar to those described might exist at many places throughout these States. In general the stream valleys are long and relatively narrow, and the streams are underlain by narrow bands of alluvial and glacial deposits that rest on and against crystalline or other bedrock. Most of the unconsolidated alluvial and glacial deposits are permeable, and are saturated with water which in most of the valleys is seemingly in hydraulic continuity with the streams. The adjoining and surrounding bedrock is relatively impermeable and can supply only small quantities of water in any one locality.

The Ipswich River Basin

The drainage basin of the Ipswich River is in Essex and Middlesex Counties in northeastern Massachusetts. The drainage area above the Geological Survey's gaging station near Ipswich is 124 square miles. The water flows into the Atlantic Ocean north of Massachusetts Bay. In the Ipswich drainage basin are several small towns, some of which obtain water directly from the Ipswich River, either for their entire supply or as standby sources, and some of which obtain their supplies from wells. There are also, of course, private homes and some small industries that obtain water from wells. A number of towns outside the drainage basin obtain water from the Ipswich, or from tributary streams and lakes. It is estimated that only a relatively small amount of the water withdrawn is available for reuse in the basin. The greatest withdrawals of ground water are those by the towns of Reading, North Reading, and Wilmington, which have a total population of about 26,000 and which altogether pump ground water at an average rate of 2.1 million gallons a day.

The area of the immediate problem is shown in Fig. 2, taken from the Salem, Reading, and Wilmington quadrangles, Massachusetts. The Ipswich River drains an area of rather low relief about half of which is underlain by bedrock covered by a thin blanket of till. The other half consists of extensive low areas that are largely swamps. The largest of these are along the river

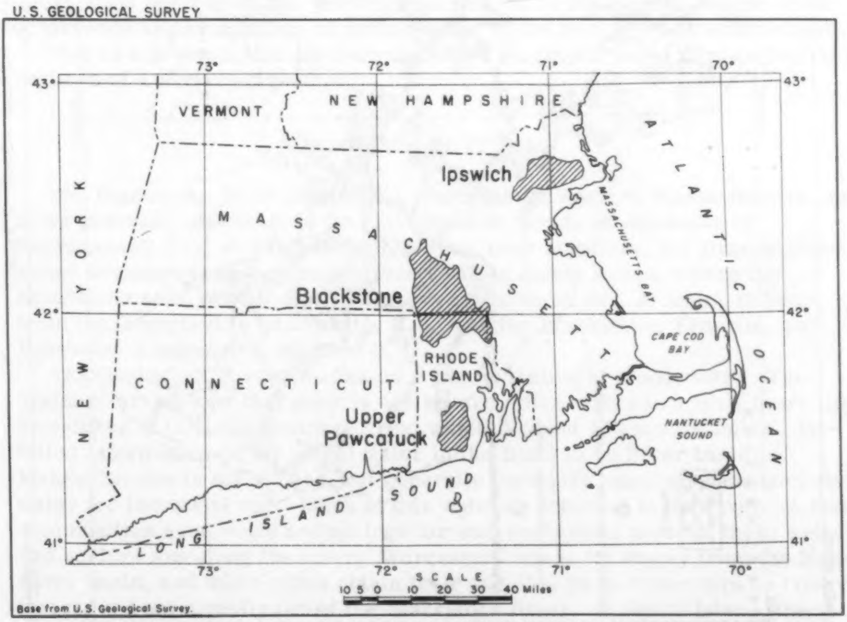

Figure 1 - LOCATION MAP SHOWING IPSWICH, BLACKSTONE AND UPPER PAWCATUCK RIVER BASINS, MASS. AND R.I.

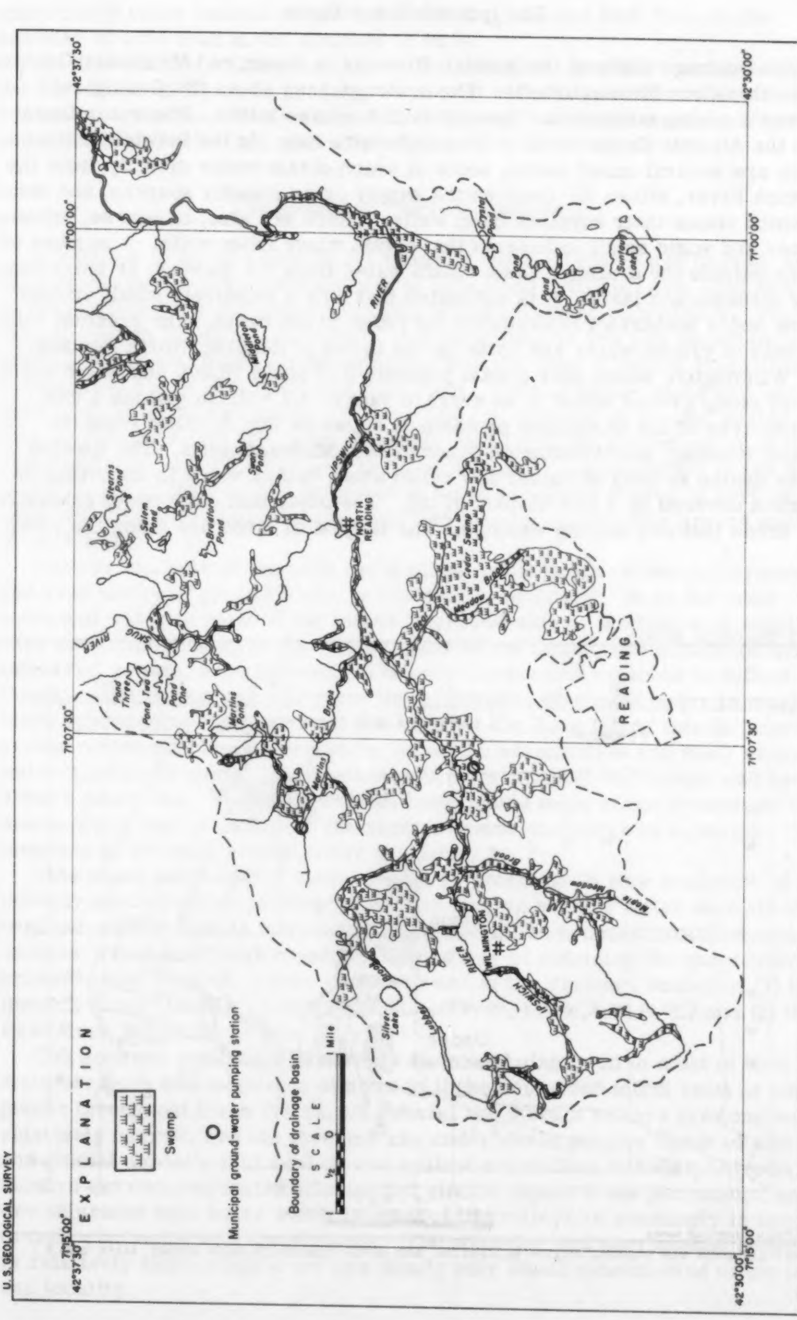

FIGURE 2 - UPPER IPSWICH RIVER BASIN, MASS.

as shown on the map. Many small to medium-sized swamps, particularly in the northeastern part, are not shown. These low areas are underlain mostly by glacial and alluvial sands and clays which may be several tens of feet thick above the bedrock. The deposits are fairly permeable and are saturated with water to within a short distance of the land surface. The larger swamps along the Ipswich River usually are flooded in the spring.

The problem in the Ipswich River drainage basin is to determine the effect on ground-water conditions of a proposed lowering and clearing of the channel of the Ipswich River. The purpose of the proposed channel work is to drain the swamps, thereby making the areas accessible for development, and to carry away storm runoff more rapidly, thereby decreasing the likelihood or frequency of flooding the presently swampy areas. Of course, many of the swamps are substantially above river level, or are in basins separated from the river by bedrock ledges, and thus would not be affected by changes in the river channel. Part of the problem is to work out the geologic conditions so as to know which swamps and lakes might be affected. In the flood plain are several installations for ground-water use, particularly those belonging to the towns of Wilmington and Reading. The ground-water reservoir that the town of Reading draws upon is in hydraulic continuity with the river and, at least at times, sustains a large part of the river flow by effluent seepage. The immediate question is as follows: If the stream channel were lowered by dredging in amounts of several feet, what, if any, would be the effect on the public water-supply wells drawing on the ground-water reservoir? The Geological Survey is assembling and will interpret the basic geologic and ground-water information bearing on this problem, as well as on the general nature of the occurrence and availability of ground water in the rest of the drainage basin.

This is a problem that involves the effect on ground water of changing the regimen of a surface stream.

Blackstone River Basin

The Blackstone River heads near Worcester in eastern Massachusetts, and flows generally southeast to the tidal Seekonk River, an extension of Narragansett Bay, at Pawtucket, R. I. One main tributary, the Branch River, whose drainage area is almost entirely within Rhode Island, enters the Blackstone near Woonsocket. The area is shown on Fig. 3* which is taken from the Georgiaville quadrangle, R. I., and the Blackstone, Franklin, and Pawtucket quadrangles, Mass. - R. I.

The Geological Survey maintains a gaging station at Woonsocket. The drainage area above that point is 416 square miles. This is a little more than four-fifths of the total drainage area, which is about 500 square miles. Detailed information on the use of water in the Blackstone River basin in Massachusetts is not at hand, but generally there are many diversions of river water for industrial use. Much of this water is returned to the river. A few communities have wells and springs for water supplies; some of these wells and springs are along the river. Worcester obtains its supply from the Nashua River basin, and other cities obtain their supplies from reservoirs on tributaries, and not directly out of the Blackstone River. In Rhode Island, there

*Geologic features on Figs. 3 and 4 are taken in large part from maps published or in preparation by Geological Survey.

are substantial diversions of water from the Branch River for use in dyeing and finishing cloth, and as sources of power for running worsted mills. In all, about 15 mgd is withdrawn from the Blackstone River and tributaries in Rhode Island, a good deal of which is returned to the river. The city of Woonsocket obtains its supply from three surface reservoirs on Crookfall Brook, a tributary of the Blackstone.

Recently, Woonsocket drilled some exploratory wells for a ground-water supply within its limits along the river but is not using this source at present. Two other towns downstream from woonsocket have wells along the river either for standby supply or for entire source.

The Blackstone River drainage basin is a hilly region in which the relatively narrow main valley lies between rounded and irregular hills. These hills are underlain by crystalline bedrock veneered by glacial till. Along the river is a flood plain underlain by alluvium and glacial outwash. This plain is not much wider than the river in many places. However, near Woonsocket and thence downstream in several reaches the plain is a quarter of a mile to half a mile wide both along the main stream and along some of its tributaries. The underlying unconsolidated deposits are 70 to 100 feet in maximum thickness.

The immediate problem is additional water for the city of Woonsocket. It is reported that the existing supply would be inadequate in the event of a dry year. Also, it is reported that water from the Blackstone River is of poor quality. Ground water is considered as a possible source of additional supply.

In the Blackstone River valley near and downstream from Woonsocket, essentially all the available ground water seems to be in the unconsolidated

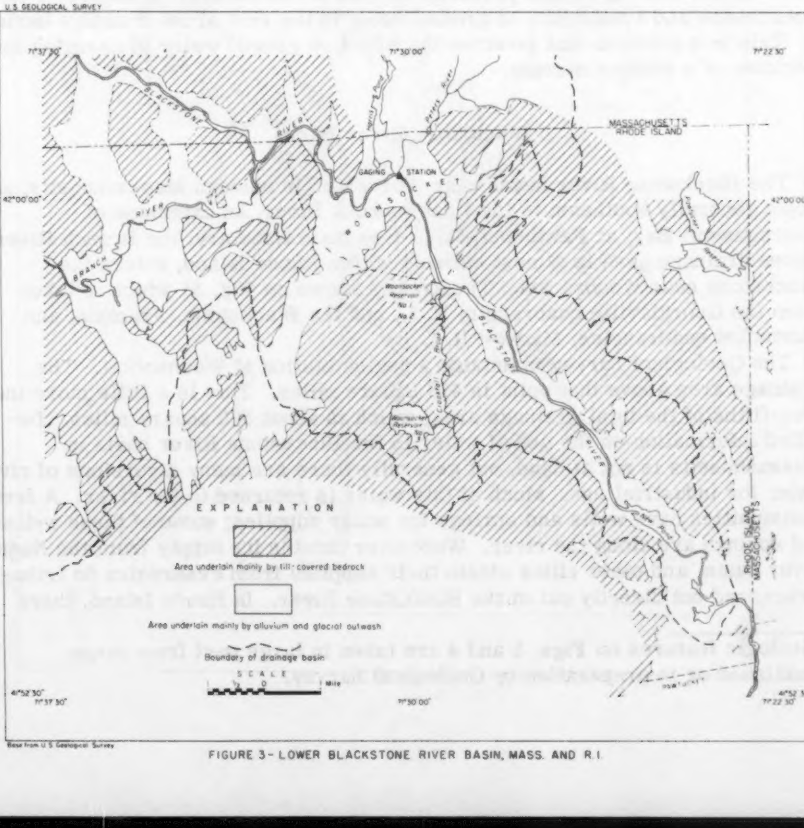

FIGURE 3 - LOWER BLACKSTONE RIVER BASIN, MASS. AND R.I.

deposits along the river. (See Fig. 3.) The water is presumably in hydraulic continuity with the stream, whose flow represents most of the available water anyway. In appraising the ground-water resource, the first thing to do is to demonstrate that there is hydraulic continuity with the river; the second is to determine the thickness, extent, and hydraulic properties of the deposits; and the third is to interpret these data and results with respect to the practicability of—essentially—pumping the river water through wells. The minimum recorded daily flow of the Blackstone River at Woonsocket was 21 cfs on August 11, 1934—about 14 mgd. In the years 1929-50, the mean discharge in August, generally the month of least flow, was 263 cfs or about 170 mgd. Many large withdrawals of ground water would depend more or less directly on streamflow for replenishment; and the river flow appears to be many times the anticipated use in the foreseeable future.

The point is that ground water is not a separate and independent resource. Assuming that the hydrologic relations are as anticipated, the practical decision would be whether it would be more feasible to develop needed water by means of wells than to develop it from surface reservoirs. Or perhaps it would be practicable to plan on taking part of the water from wells and part from surface reservoirs.

Similar problems are developing in Massachusetts, where municipal supplies in some areas are taken from lakes. These lakes probably are expressions of the water table and are essentially continuous with the ground-water reservoir. In many places it may prove desirable to plan on deriving municipal supplies from wells, and using the lakes for recreational purposes.

Pawcatuck River Basin

The Pawcatuck River is mainly in southwestern Rhode Island but has several small tributaries in Connecticut. The total drainage area above Westerly, R. I., where the river enters Long Island Sound, is 295 square miles. This paper is concerned, however, with the portion of the drainage basin above the Geological Survey's gaging station near the junction with the Wood River, and mainly in the Kingston and Slocum quadrangles, R. I. This portion comprises the basins of the Usquepaug and Chipuxet Rivers, which unite in Great Swamp to form the Pawcatuck River. (See Fig. 4.) This basin is referred to as the upper Pawcatuck River basin; its drainage area is about 100 square miles.

At present only small amounts of water are used in this upper drainage basin. Industries use minor amounts of both surface water and ground water; and public supplies come entirely from wells. The total use is between 2 and 3 mgd.

The upper Pawcatuck River basin is a relatively low-lying area characterized mostly by low slopes and plains along the streams. The areas between the stream valleys are characterized by low hills, all less than 500 feet above sea level. These are underlain by crystalline rock with the usual discontinuous blanket of till. This bedrock terrane occupies about half the drainage area.

The plains along the streams are underlain by unconsolidated deposits, mainly of glacial origin, that attain a maximum known thickness of about 150 feet. These deposits are saturated with ground water generally to within 10 feet or less of the land surface. This ground water is hydrologically connected with many swamps and lakes, of which Worden Pond and Great Swamp, having a combined extent of 6 or 8 square miles, are the biggest.

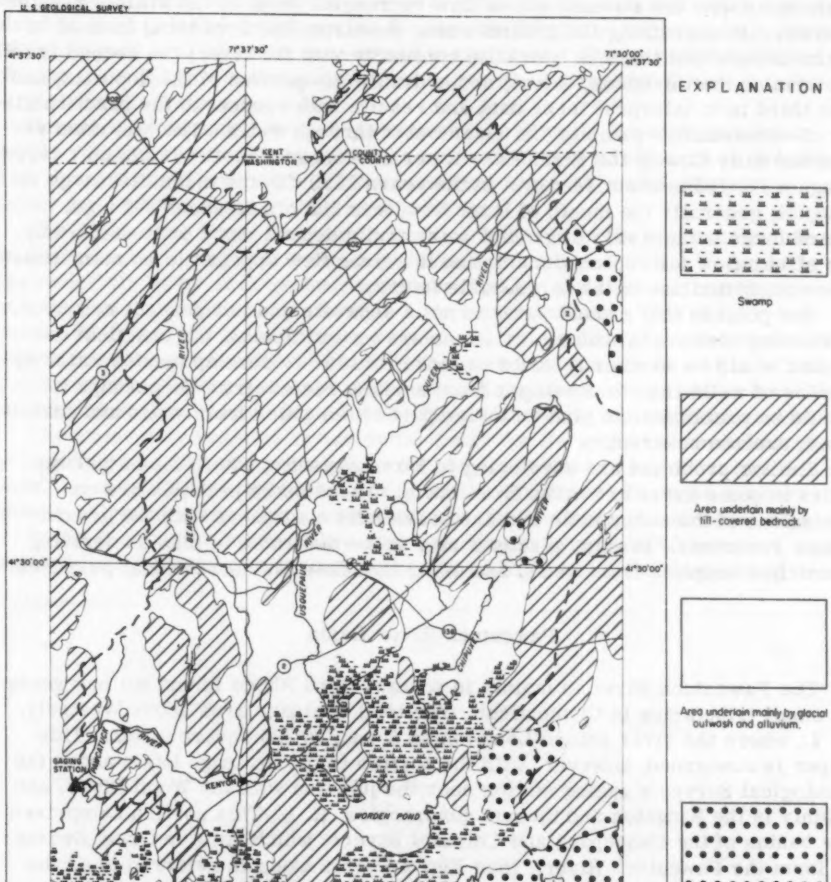

FIGURE 4 UPPER PAWCATUCK RIVER BASIN, R.I.

The streams flow across this area of plains, lakes, and swamps, transporting overland runoff, and also ground water that discharges into them. Near the west edge of the Kingston quadrangle, above Kenyon, the Pawcatuck proper flows between two hills composed of bedrock. Here the glacial deposits are probably thin, and not more than a quarter of a mile wide. In the narrows, essentially all the ground water flow is forced to the surface and appears in the stream channel together with the surface discharge. This flow is essentially all the water that leaves the basin in liquid form, though much of course is evaporated and transpired.

In the upper Pawcatuck River basin, the State of Rhode Island is interested in ascertaining the location and amount of ground water available for possible industrial development along the main line of the New York, New Haven, and Hartford Railroad. Mapping the geology, interpreting information from wells already drilled, and determining the nature, thickness, and extent of water-bearing deposits will locate sources of ground water, and may indicate the potential production of individual wells. These techniques, however, will not result in determining the total amount of ground water available for development. This has to be done by quantitative methods, of which measurements of the surface water are an important part.

Of the water that enters the basin as rain and snow, part percolates below ground to the ground-water reservoirs, and part moves directly into the streams. The part in the streams moves across the basin as surface flow; the part in the ground-water reservoir is discharged partly by evapotranspiration, probably mostly in the Worden Pond-Great Swamp area, and partly by effluent seepage into the Pawcatuck River in and above the narrows near Kenyon.

On the principle that inflow equals outflow plus or minus change in storage, the total discharge near Kenyon is a measure of the amount of water entering the basin, less the evapotranspiration within the basin. Assuming that it is impracticable to intercept and store the overland runoff, mainly flood water, the available water is the portion of the total outflow that results from groundwater runoff alone. The amount of this runoff can be estimated from yearly hydrographs by separating out the purely or largely overland runoff. An example of such a separation was given in a report on the geology and groundwater conditions of the Kingston quadrangle, Rhode Island, published by the State.

If, in the process of pumping the ground water from an area such as the Worden Pond-Great Swamp area, the water level were lowered appreciably, some additional water might be made available through reduction of evapotranspiration losses in that area; and some added storage space might be created in which overland runoff might be trapped.

It is difficult to determine these quantities closely. The main point is that, by measuring the surface runoff, it is possible and in some places practicable to estimate ground-water supplies quantitatively.

CONCLUSION

Thus, for specific and quantitative investigations of ground-water resources, it is often desirable, if not necessary, to consider ground water in conjunction with surface water—both as interrelated parts of the same resource.

ACKNOWLEDGMENTS

The writer thanks his colleagues in the Geological Survey, W. B. Allen, O. M. Hackett, C. E. Knox, and S. M. Lang, for review and constructive criticism of the manuscript.

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PRESSURE CHANGES AT OPEN JUNCTIONS IN CONDUITS

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Ernest T. Smerdon;[3] and Herbert G. Bossy,[4] M. ASCE

SYNOPSIS

This paper contains the results of an analytical and experimental investigation of the effects of open junctions on the magnitude of pressure changes in closed conduits flowing full. Junctions of rectangular, square, and round plan were studied.

Data concerning the performance of such junctions have been extremely meager in the past and designs have therefore been based on rather arbitrary procedures. The present paper furnishes data which afford the means for a rational hydraulic design of these structures.

INTRODUCTION

While the investigation reported herein was designed to furnish data specifically suited to the design of storm drain systems, it is believed that the results have a somewhat wider range of applicability. Hence, no attempt is made in this paper to restrict the discussion to any particular field of application.

Past practice has ascribed to open junctions in pipe systems an arbitrary yet small head loss. The authors have, however, determined that losses at such junctions may indeed be of an important magnitude. As a result, new methods of analysis have been proposed and old ones modified so as to provide a more realistic basis for design.

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Because the designer is interested primarily in the variation of pressure throughout the system rather than in the variation of total head, the results are presented in terms of the changes in piezometric head prevailing at these junctions in preference to the more customary total head loss coefficients.

General

The shape of junction of primary interest to one of the sponsoring agencies, and therefore the one studied in greatest detail, was a rectangular one having a width-to-length ratio of 1 to 2.5. As a consequence, the analyses and tests applicable to such structures are presented first. A subsequent section extends the discussion to the more general cases of square and round junctions and, in addition, correlates these to the rectangular.

In order to identify each pipeline and the conditions therein a systematic subscript notation has been employed. The number "1" is reserved for conditions in the downstream pipe and the following larger numbers are assigned in order to conditions in the pipes met in turn in moving from the downstream pipe in a clockwise manner in the plan view.

So that the effect of the junction might be properly delineated, the established piezometric head lines in the various pipes were projected to the center of the junction so that any pipeline resistance losses would automatically be excluded from the pressure change attributable to the junction. The method of determination of the pressure change is described in detail in the following section.

Experimental Methods

Test Equipment

A general isometric view of the test equipment for a typical junction is presented in Fig. 1. Metered flows were supplied to Lucite model pipelines which were in turn connected to the model junction. The junction itself was constructed of Plexiglas plate.

Each of the header tanks indicated in Fig. 1, was supplied with a series of baffles, a rounded inlet for the pipeline, and a set of straightening vanes within the pipeline itself. The purpose of each of these appurtenances was to aid in obtaining fully established flow in as short a distance along the pipeline as possible. As a further means of ensuring normal velocity distributions in the pipes, all lines were made as long as was compatible with the physical dimensions of the laboratory. Lengths of at least 25 pipe diameters were employed, and in every case but one lengths of more than 40 diameters were used. Such lengths have been stated(1) to be sufficient for the development of fully established flow.

The outlet box and tail gate shown in Fig. 1 were provided for two purposes; viz., (1) to force all the pipes in the system to flow full, and (2) to permit variation of the depth of water in the junction.

Generally seven piezometers were connected to each of the model pipelines to define the slope of the piezometric head lines. These piezometers were concentrated most heavily at the downstream end of each pipe so that the slope of the piezometric head line in established flow might be determined. In addition a collection ring was attached to the model junction with several taps

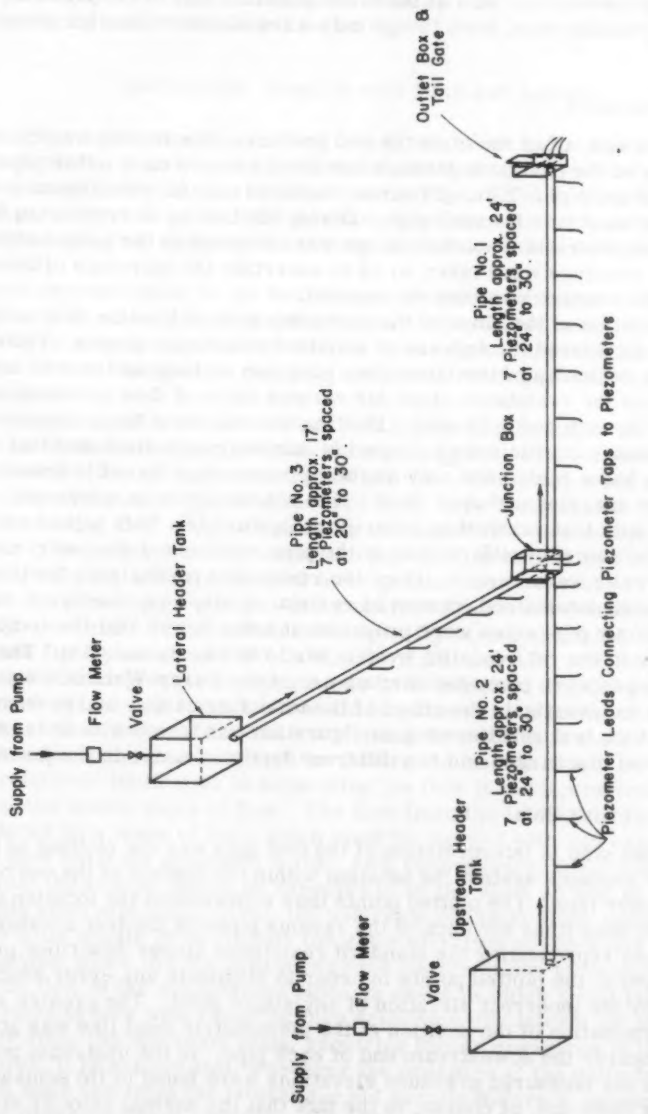

Fig. 1. General layout of model system.

through the walls of the junction so that a measure of the average elevation of the free surface in the junction might be obtained.

Four sizes of Lucite pipe were used for the model pipelines; namely, 3.00, 3.75, 4.75, and 5.72 inches in inside diameter. Use of these sizes permitted variation of the relative size of the three pipelines and of the pipe size relative to the junction size, even though only a few absolute junction sizes were employed.

Testing Procedure

In practically all of the tests the end product of the testing was the determination of the change in piezometric head between each inflow pipe and the downstream pipe. This, of course, required careful establishment of the piezometric head line for each pipe. During the testing of certain configurations a considerable amount of surge was observed in the piezometers, in which case readings were taken so as to ascertain the extremes of this fluctuation and the average was then determined.

Determination of the slope of the piezometric head line for flow in the model was facilitated through use of standard resistance slopes. Tests were made at the beginning of the laboratory program on long sections of the model pipe to define the resistance slope for various rates of flow in two sizes of pipe, the 3.00-inch and 5.72-inch. Plotting the results of these tests on the usual resistance coefficient vs. Reynolds number graph disclosed that the Lucite pipe had a resistance only slightly greater than that of hydraulically smooth pipe carrying turbulent flow. This conclusion is in substantial agreement with that reached by other investigators.(2) This higher value was caused mainly by non-uniformities in the pipe section and diameter, even though the walls were smooth. Once the resistance coefficients for these two sizes had been determined, curves of resistance slope vs. discharge rate for each of the four pipe sizes were prepared, it being known that the temperature of the water in the recirculating system would be nearly constant. These resistance slopes were computed through use of the Darcy-Weisbach equation.

In order to investigate the effect of the total rate of flow and to ensure the accuracy of the test results, each configuration was tested with at least two different total discharges and two different depths of water in the junction.

Treatment of Test Data

The initial step in interpretation of the test data was the plotting of the piezometer readings against the location within the system of the corresponding piezometer taps. The plotted points thus represented the location of the piezometric head lines for each of the various pipes of the test arrangement. Straight lines representing the standard resistance slopes described previously were fitted to the plotted points in order to eliminate any error which might be caused by the incorrect elevation of any single point. The greater weight in the determination of the position of the piezometric head line was accorded the points nearer the downstream end of each pipe. In the upstream portion of the pipes the measured pressure elevations were found to lie somewhat above these lines due, of course, to the fact that the normal velocity distribution had not been fully developed in this section.

The determination of pressure changes across the junction required the measurement of the vertical distance between the plotted upstream and downstream pressure lines, each projected to some common vertical plane or line.

For convenience in design the point of intersection of these pipe centerlines at the centerline of the box was selected as the point of reference. These points are called "branch points" after McNown.(3)

For convenience of application to design, the measured pressure changes were plotted in their ratio to the mean velocity head in the downstream pipe thus producing a dimensionless coefficient.

Rectangular Junction with Main and Lateral

This pipe arrangement consisted of a through line and a pipe aligned perpendicularly to this main or through line entering the narrow side of the junction as indicated in plan in the insets in Figs. 2 and 3. The flowlines of all the pipes were flush with the junction floor since an early series of tests indicated negligible differences between the cases of flowline and crownline alignment.

Pipes perpendicular to the downstream pipe occur frequently in the discussions of this configuration and of those which follow; hence, for simplicity they will be referred to subsequently as "laterals". Laterals, when used, carried flow to the junction and only one downstream pipe was included in any of the systems.

In this series of tests the downstream pipe generally had a flush, square-edged entrance at the junction box, although a few runs were made to ascertain the effect of other entrance conditions. In these additional tests rounded and projecting entrances of types discussed later were investigated.

In combining flow such as that being discussed, the downstream pipe should ordinarily be larger than any upstream pipeline, so that values of the diameter ratios D_1/D_2 and D_1/D_3 equal to or greater than unity were the only ones tested. Only two sizes of pipe were employed for the downstream pipe; viz., the 4.75-inch and 5.72-inch diameters. D_1/D_2 values of 1.00, 1.20, 1.58 and 1.91; and D_1/D_3 values of 1.00, 1.20, 1.53, 1.58, and 1.91 were tested.

For this arrangement of pipes it was to be expected that the momentum of the flow from the upstream main would be preserved to some extent in crossing the junction and would aid in maintaining flow in the downstream pipe. On the other hand, the momentum of the flow in the lateral should be anticipated to be relatively ineffective in supporting the flow in the downstream pipe, at least at the lesser rates of flow. The flow from the lateral thus should be considered as a mass of fluid which must be merged with and carried along by the flow from the upstream main. As a consequence, the pressure at the exit from the lateral should be expected to be the same as that at the exit from the upstream main. Also, the upstream pressure lines projected to the branch point should be at roughly the same elevation as the water surface in the junction, since the box pressure is the source of one force which maintains the flow in the downstream pipe. In cases in which small laterals carry large rates of flow with resulting high velocities, the lateral jet should disrupt the flow from the upstream main, requiring higher pressures in the junction. Moreover, it is evident that, whenever the pressure at the exits from the upstream main and from the lateral are not greatly different, an analysis for K_2, the pressure change coefficient for the upstream main, will also be applicable with satisfactory accuracy to the valculation of K_3, the pressure change coefficient for the lateral. This situation should occur when a relatively small proportion of the total flow in the system enters from the lateral.

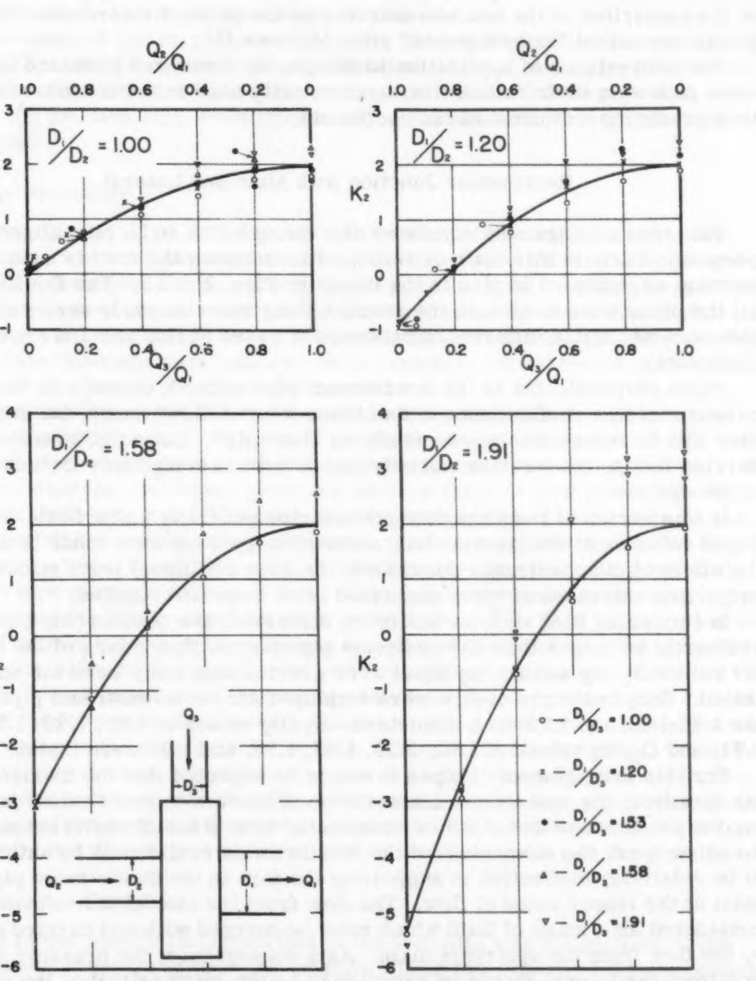

Fig. 2. K_2 for in-line system with 90° lateral.

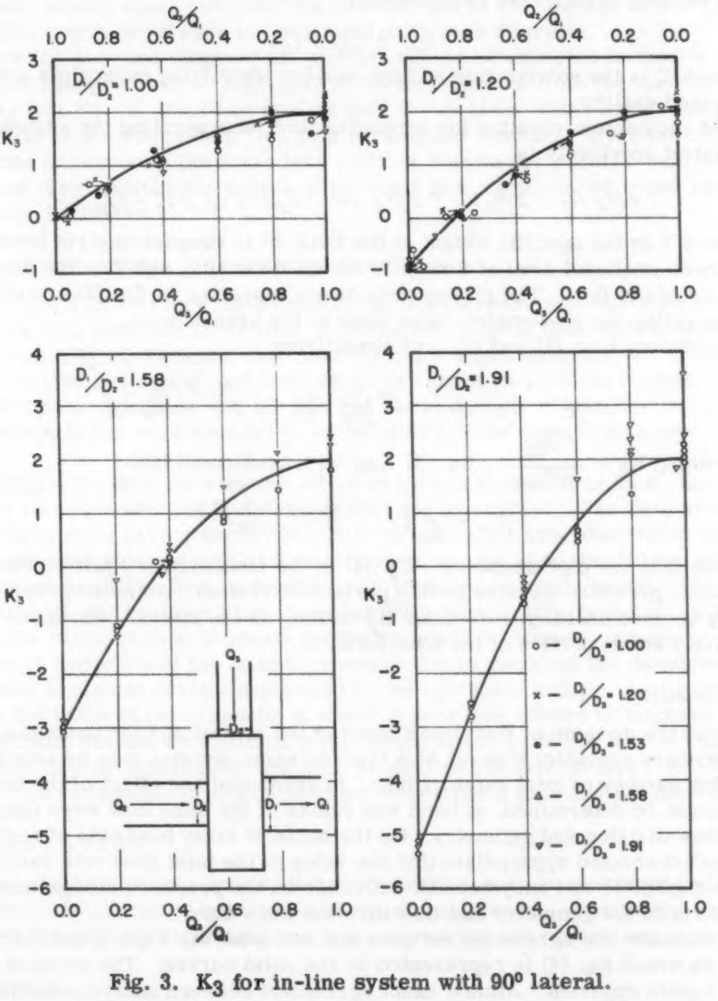

Fig. 3. K_3 for in-line system with 90° lateral.

Theoretical Analysis

A method of analysis involving the momentum concept is useful in this case. As noted earlier, it seems logical to assume that the flow from the upstream main contributes the only momentum effective in the direction of the downstream flow, and that the flow from the lateral furnishes an added mass to be carried into the downstream pipe by the force of the box pressure. An analysis based on this foundation should be expected to be least accurate in those cases in which the lateral efflux velocity is high enough to cause the jet from the lateral to impinge on or penetrate that from the upstream main, thereby causing both to be deflected and diffused with an attendant increase in the level of pressure in the box required to maintain flow.

Continuity in this case requires that

$$Q_1 = Q_2 + Q_3 \tag{1}$$

in which Q is the volume rate of flow, and the identifying subscripts are as discussed earlier.

The momentum equation for expanding flow incorporating the assumptions described previously is

$$\gamma h_2' A_2 - \gamma h_1' A_1 + \gamma h_2' (A_1 - A_2) = Q_1 \rho V_1 - Q_2 \rho V_2 \tag{2}$$

in which γ is the specific weight of the fluid, h' is the piezometric head, A is the cross-sectional area of flow, ρ is the fluid density, and V is the mean velocity of the flow. The piezometric heads appearing in Eq. (2) are obtained by projecting the piezometric head lines to the branch point.

Combining Eqs. (1) and (2), and simplifying

$$h_2 = h_2' - h_1' = 2 \frac{V_1^2}{2g} \left[1 - \left(\frac{Q_2}{Q_1}\right)^2 \left(\frac{A_1}{A_2}\right)\right] \tag{3}$$

Defining $K_2 = \frac{h_2}{V_1^2/2g}$, Eq. (3) may be transformed into

$$K_2 = 2\left[1 - \left(\frac{Q_2}{Q_1}\right)^2 \left(\frac{D_1}{D_2}\right)^2\right] \tag{4}$$

wherein D is the pipe diameter. Eq. (4) is the fundamental equation for determining pressure changes in this particular system. As noted previously, it may be used unchanged to obtain K_3 so long as the lateral introduces only a relatively small portion of the total flow.

Test Results

While the division of flow rates between the lateral and the upstream main is a primary variable, it is obvious that the same division may be obtained with any number of total rates of flow. In order that the effect of the total rate might be determined, at least two values of the total flow were tested for each flow division and geometry. On the basis of many hundreds of tests the conclusion seemed appropriate that the value of the total flow rate exerted only a negligible and unsystematic influence on the pressure change coefficients once the geometry and flow division were fixed.

To examine the agreement between test and analysis Figs. 2 and 3 are provided on which Eq. (4) is represented by the solid curves. The trend of the data is quite apparent. Almost exact agreement between analysis and test exists from $Q_3/Q_1 = 0$ to the flow division at which K_2 or K_3, as the particular case may be, is zero. This, of course, occurs when the momenta of the flow in the upstream and downstream mains are equal, since the quantity $\left(\frac{Q_2}{Q_1}\right)^2 \left(\frac{D_1}{D_2}\right)^2$ in Eq. (4) is the ratio of these momenta. It is further evident that beyond the flow division for which K is zero the equation gives a fairly good average value of the test points. When the lateral and downstream pipes are of relatively the same size, the agreement extends nearly through the entire range of flow divisions. Only when small laterals are employed (with consequently large values of the lateral momentum) does the deviation become appreciable. As a consequence Eq. (4) can be depended upon when $Q_3/Q_1 \leq 0.4$ for any geometry and for all values of Q_3/Q_1 when the lateral and downstream pipe

sizes are nearly equal, provided the junction box is rectangular and of considerable length in relation to the downstream pipe diameter.

Since the size and shape of the junction should be expected to modify the pressure changes when a large part of the total flow is from the lateral, it is obvious that Eq. (4) should be applied only to junctions which are geometrically similar to the one tested; i.e., to a rectangular junction. Even for these junctions the equation produces less reliable results for smaller size laterals carrying disproportionate shares of the total flow. Square and round junctions are discussed later.

Effect of Downstream Pipe Entrance Conditions

The effect of entrance conditions at the downstream pipe was also investigated for this system. In addition to the square-edged, flush entrance utilized in the majority of the tests, two other entrance types were tested. One was a square-edged projecting pipe with a wall thickness of 1/12 the downstream pipe diameter extending into the box 1/4 the box width. The other was a flush entrance with the edge rounded to a radius of 1/8 the downstream pipe diameter.

In Fig. 4 the data showing the effect of entrance conditions on K_2 and K_3 for the two pipe-size combinations tested are assembled. The re-entrant condition is seen to have a negligible effect on lateral or upstream-main pressure throughout the entire range of flow divisions, except when small laterals carry almost all the flow, a condition not likely to be encountered. The rounded entrance, on the other hand, does exhibit a measurable reduction in K_2 and K_3 in the range of flow divisions producing positive pressure changes. The degree of reduction of the lateral pressure due to rounding the downstream entrance is almost exactly duplicated in the upstream main.

On the basis of these results it would appear that efforts to improve the downstream entrance conditions can produce minor benefits, but only in the range of flow divisions involving high proportions of lateral flow. It is also clear that reasonable projections of the downstream pipe are not detrimental.

Through Line Only

Many tests were run on a rectangular junction with an upstream and downstream main aligned in plan and with no lateral present. Two cases were recognized in these investigations; viz., expanding and contracting flow.

Expansions

With an expansion of flow the analysis leading to Eq. (4) applies in this case also, except that $Q_1 = Q_2$. Thus

$$K_2 = 2\left[1 - \left(\frac{D_1}{D_2}\right)^2\right] \tag{5}$$

Contractions

The primary difference in the analysis of contractions as compared with that for expansions involves the application of the momentum principle between the contracted section just inside the downstream pipe and a section further downstream at which the flow has expanded to again fill the pipe. Losses due to boundary separation occur almost exclusively in the zone

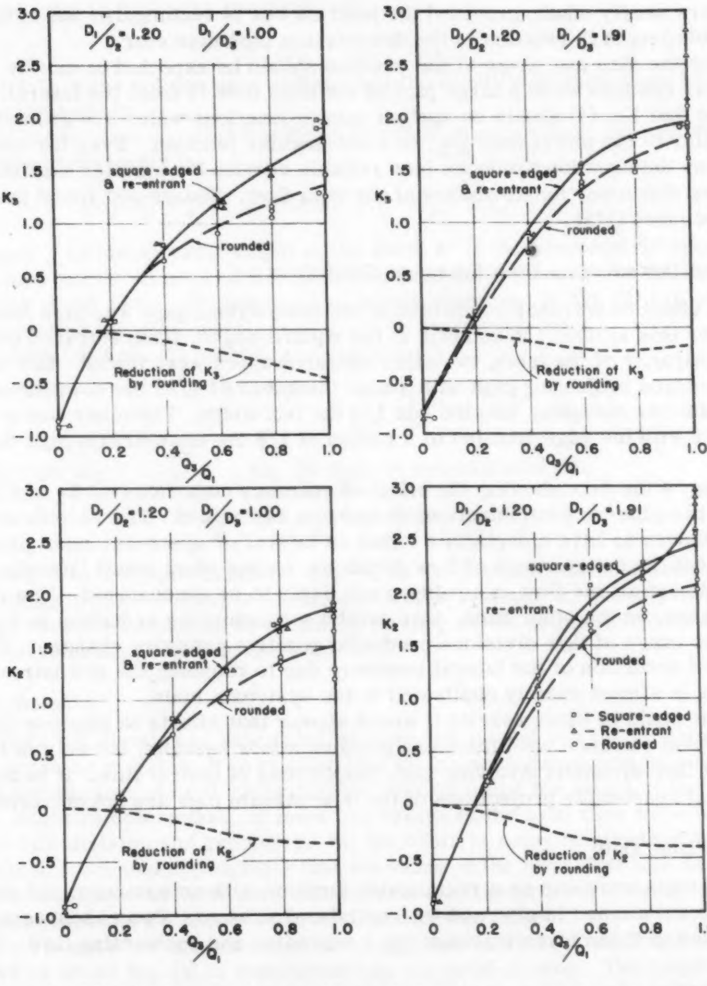

Fig. 4. Effect of projecting and rounded entrances to downstream pipe.

following the section of greatest contraction of the flow (where conditions are denoted by the subscript c). As in the case of expansions, inclusion in the final equation of losses due to surface resistance is circumvented by projecting the piezometric head lines to the branch point.

Continuity requires that

$$Q_1 = A_1 V_1 = Q_c = A_c V_c = Q_2 = A_2 V_2 \tag{6}$$

The momentum principle can be applied between sections c and 1 (downstream) to yield

$$\gamma h_c' A_1 - \gamma h_1' A_1 - R = Q\rho (V_1 - V_c) \tag{7}$$

in which R is the boundary shear force in the pipeline between sections c and 1 and the (h')'s in this case are the piezometric heads at sections c and 1. Eq. (7) is true by virtue of the fact that the piezometric head at section c must be constant completely across the section of area A_1, since the streamlines of the submerged jet are parallel there. Eq. (7) combined with Eq. (6) reduces to

$$h_c' - h_1' - \frac{R}{\gamma A_1} = \frac{1}{gA_1}\left[V_1^2 A_1 - V_c^2 A_c\right] = 2\frac{V_1^2}{2g}\left[1 - \frac{1}{C_c}\right] \tag{8}$$

in which C_c is the ordinary contraction coefficient.(4)

The Bernoulli or energy equation written between sections 2 and c, in which region a small resistance loss, H_f, occurs, gives rise to

$$h_c' + \frac{V_c^2}{2g} = h_2' + \frac{V_2^2}{2g} - H_f \tag{9}$$

Combination of Eqs. (8) and (9) leads to

$$h_2' - h_1' - \frac{R}{\gamma A_1} - H_f = \frac{V_1^2}{2g}\frac{1}{C_c^2} - \frac{V_1^2}{2g}\frac{A_1^2}{A_2^2} + \frac{V_1^2}{2g} + \frac{V_1^2}{2g}\left[1 - \frac{2}{C_c}\right] \tag{10}$$

If the piezometric head lines are projected to the branch point, the left side of Eq. (10) is approximately h_2. The only discrepancy is the difference in the resistance slopes for the two pipes multiplied by the distance between the junction centerline and the contracted section. This is obviously of secondary importance.

Inserting the diameters D_1 and D_2, and $K_2 = \dfrac{h_2}{V_1^2/2g}$ into Eq. (10) results in

$$K_2 = 1 - \left(\frac{D_1}{D_2}\right)^4 + \left(\frac{1}{C_c} - 1\right)^2 \tag{11}$$

Eq. (11) is the desired formula for the pressure change coefficient for contractions in a two-pipe system.

Test Results

Either of Eqs. (5) or (11) should be applicable to the analysis of flow in equal-size pipes. Both equations yield zero for K_2, while the experiments indicate a positive but small value for this coefficient. It should be pointed out that in the derivation of the two equations mentioned, the junction width, b, was considered to be negligible; i.e., the entire unit was assumed to behave as a sudden expansion or contraction, as the case may be. Still it seems reasonable that at some large value of the parameter b/D_2 the jet will not remain essentially intact while crossing the junction, and a consequently larger loss will occur. This increase is due in part to viscous shear, but results primarily from the impingement of the spreading jet on the wall surrounding the entrance to the downstream pipe.

To test this hypothesis configurations having values of b/D_2 up to 3.33 were investigated with equal size upstream and downstream pipes. Fig. 5 presents the results of these tests. The effect of increasing junction width seems to be a fairly uniform increase in K_2, although it appears likely that some limiting value will be attained as the junction box is further enlarged. Definition of this limit was not attempted since the ratio of the distance across the box to the inflow pipe diameter was already near the upper end of the range likely to be encountered in practice.

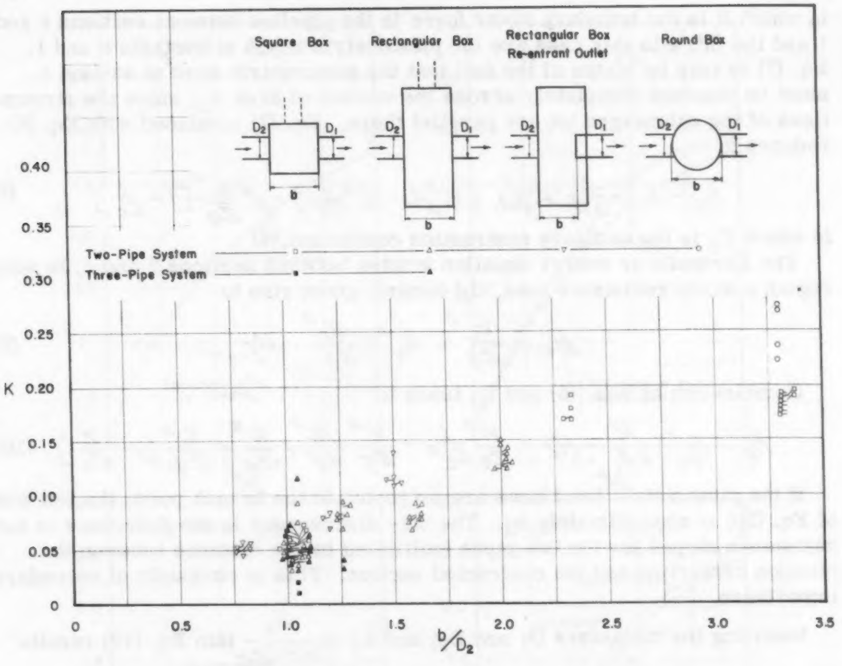

Fig. 5. Effect of junction width for equal-size pipes.

A series of tests was made with the available model pipes providing several ratios of upstream to downstream pipe sizes, and thus various degrees of expansion of the flow across the rectangular junction first described. The test results were compared to the theoretical analysis embodied in Eq. (5).

In Fig. 6, Eq. (5) is shown as a solid curve in the area of negative changes of pressure and the pressure change coefficients derived from the test data are plotted and suitably identified. Obviously the conformity between analysis and experiment is quite satisfactory. The large negative values are easily explained if it is recalled that the coefficients represent pressure changes and not changes in total head. Thus negative coefficients corresponding to pressure rises are to be expected whenever the fluid is caused to decelerate. In the present case this is brought about by the increase in pipe diameter on the downstream side of the junction.

Similar tests were made with various degrees of contraction of the flow across the same junction, and the results compared to the theoretical analysis expressed in Eq. (11).

In Fig. 6, Eq. (11) is shown as a dashed curve in the area of positive changes of pressure. The experimental pressure change coefficients for this case are also plotted for comparison. It is seen that the conformity between analysis and experiment is also satisfactory for contractions of flow.

Probably the most significant conclusion to be reached from the tests just described is the absence of effect of the box itself—at least for the range of the parameter b/D_2 tested. Thus, it would be expected that variations of construction details at the junction would not be effective in modifying its characteristics. However, the more common construction details were investigated.

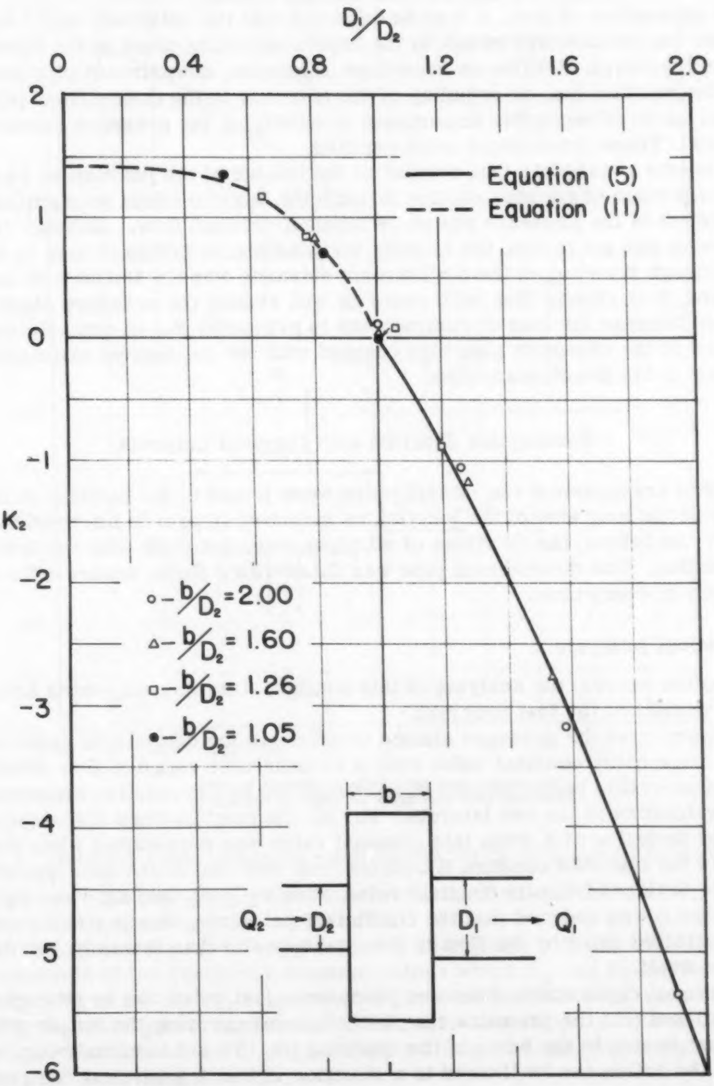

Fig. 6. Results for two-pipe in-line system.

As was mentioned earlier, the vertical positioning of the pipe centerlines is immaterial so long as they are parallel and are aligned between limits in which either their crownlines or flowlines coincide.

In expansions of flow, it may be expected that the relatively short distances across the junction will result in the expansion taking place in the downstream pipe. Therefore, flowline or crownline alignment, downstream pipe projecting into the junction box, or rounding of the entrance to the downstream pipe should all be of negligible importance in modifying the pressure change coefficients. These conclusions were verified.

It seems reasonable that shaping of the bottom of the junction so as to continue a portion of the pipe section through the box can effect no significant improvement of the pressure change in straight-through flow. Although this inference was not tested, the latitude for possible reduction of loss is small.

Although rounding of the downstream entrance was not tested with contracting flow, it is evident that such rounding will reduce the pressure change coefficients because the loss at contractions is primarily due to constriction of the flow at the change in pipe size coupled with the subsequent expansion of the flow in the downstream pipe.

Rectangular Junction with Opposed Laterals

In this arrangement two lateral pipes were joined to the junction at the center of the long side of the junction as indicated in plan in the inset in Fig. 7. As before, the flowlines of all pipes were set flush with the bottom of the junction. The downstream pipe was fitted with a flush, square-edged entrance in every case.

Theoretical Analysis

As often occurs, the analysis of this configuration was suggested and guided by the results of the test program.

In every case the pressure change coefficients for one lateral appeared to have a reasonably constant value over a considerable range of flow divisions. The extent of this range seemed to be controlled by the relative magnitudes of the velocities in the two laterals. For all the combinations investigated, the first deviation of K from this constant value was perceptible when the velocity in the line with constant K became less than that in the line opposite. Thus K_2 deviated from its constant value when $V_3 \geq V_2$, and K_3 when $V_2 \geq V_3$. From this it was deduced that the coefficient pertaining to a particular pipe was controlled more by the flow in the pipe opposite than it was by the flow in the pipe itself.

A rational explanation of the two phenomena just noted can be attempted if it is realized that the pressure rise in the lateral carrying the lesser-velocity flow must be due to the force of the opposing jet. To aid in visualizing the forces, the action can be likened to a situation in which a vertical flat plate divides the junction box in a direction normal to the centerlines of the incoming laterals in such a way as to deflect the two jets. The resultant of the distributed pressure along this plate due to one of the jets may be considered to be linearly proportional to the jet velocity head.[5] The net force exerted on the plate is the difference between the forces exerted by the two jets and may be considered as transmitted to the fluid and eventually to the wall of

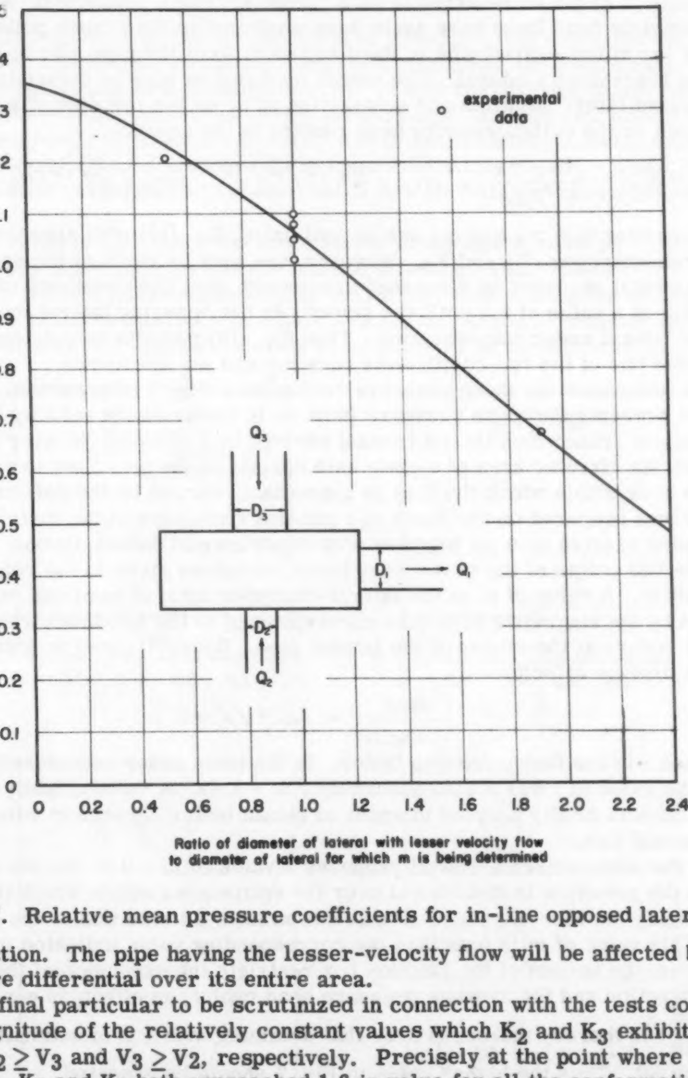

Fig. 7. Relative mean pressure coefficients for in-line opposed laterals.

the junction. The pipe having the lesser-velocity flow will be affected by this pressure differential over its entire area.

The final particular to be scrutinized in connection with the tests concerns the magnitude of the relatively constant values which K_2 and K_3 exhibited when $V_2 \geq V_3$ and $V_3 \geq V_2$, respectively. Precisely at the point where $V_2 = V_3$, K_2 and K_3 both approached 1.6 in value for all the conformations investigated in this series of tests. Thus 1.6 might be taken as a "base" value for the pressure change coefficients.

An attempt is made in the following to combine the significant facts just outlined into a mathematical analysis of the situation.

Utilizing a notation comparable to those previously employed, and assigning the higher velocity to lateral pipe 2, an expression embodying the fundamental deduction from the experiments may be written

$$h_3' - h_2' = m_2 \frac{V_2^2}{2g} - m_3 \frac{V_3^2}{2g} \tag{12}$$

in which m might be referred to as a "mean pressure coefficient" and the piezometric head lines have again been projected to the branch point. The value for m for a given pipe is fixed by the ratio of the pipe size to the size of the low-velocity lateral. The means by which m may be determined will be discussed later. Addition and subtraction of h_1' on the left side of Eq. (12) and division by the outfall velocity head results in the equation

$$K_3 - K_2 = \frac{h_3 - h_2}{V_1^2/2g} = m_2 \left(\frac{Q_2}{Q_1}\right)^2 \left(\frac{D_1}{D_2}\right)^4 - m_3 \left(\frac{Q_3}{Q_1}\right)^2 \left(\frac{D_1}{D_3}\right)^4 \tag{13}$$

Assuming that m_2 and m_3 can be estimated, Eq. (13) still appears to contain two unknowns, K_2 and K_3. However, use may be made of the second fundamental observation discussed previously; viz., the constancy of the coefficients at a value of 1.6 until the velocity in the opposing lateral exceeds that in the lateral under consideration. Thus Eq. (13) permits calculation of the variable one of the two coefficients once m_2 and m_3 are known.

To determine the mean pressure coefficients Fig. 7 is presented. This figure presents the mean pressure (written in terms of the velocity head in the lateral from which the jet issues) exerted by a circular jet over any nearly concentric circular area of a plate held normal to the jet. This is restricted to the zone within which the flow is appreciably curved by the deflection. The curve was prepared on the basis of a general knowledge of the distribution of pressure exerted by a jet together with experimental substantiation.

The end points of the curve were based on values given in the reference literature. A value of m at the lateral-diameter ratio of zero can be approximated by the stagnation pressure corresponding to the maximum velocity, which occurs at the center of the lateral pipe. Rouse[6] gives an equation for the centerline velocity

$$\frac{V_{max}}{V_{mean}} = 1.43\sqrt{f} + 1$$

in which f is the Darcy friction factor. In the tests under consideration the average value of f was 0.018; accordingly m = 1.42. A value slightly lower than this was finally adopted in order to obtain better agreement with the experimental data.

At the other extreme Powell proposes a value of m = 0.33 for the case in which the pressure is distributed over the entire area within which the flow is sensibly curved. He suggests that this area is 6 times that of the issuing jet. This value of m is less than the corresponding value indicated in Fig. 7. However, the bottom of the junction box restricts the expansion of the jet in that direction and the average pressure head (which, in effect, is equivalent to $m\frac{V^2}{2g}$) is thereby increased over that obtaining with a symmetrically expanding jet. Thus the higher value in Fig. 7 should be expected.

For a diameter ratio of unity, m must be approximately equal to the kinetic energy correction factor for the lateral flow, which in this case is roughly 1.05.

In order to determine the shape of the curve between these three values, recourse was had to the experimental data. Substitution of the test results into Eq. (13) for a given geometry and with all the flow through one lateral yielded the value of m for that lateral. The results of the calculations from the experimental data and the pseudo-theoretical curve are presented in Fig. 7.

Test Results

Eq. (13) is plotted in Fig. 8 together with the applicable experimental points. The correlation is clearly very good throughout the entire range of pipe-size ratios and flow divisions tested. In this regard it is appropriate to note that the one set of tests involving unequal-size laterals represents conditions as greatly different from equal-size laterals as was possible with the equipment available. Since Eq. (13) is substantiated for $D_2/D_3 = 0.524$, 1.00, and 1.91, it is reasonable to assume that it is also valid for intermediate conditions.

To be noted in Fig. 8 are the large values of the coefficients accompanying substantially different velocities in the two laterals. Thus when unequal velocities in the two laterals are anticipated or may occur, adoption of this arrangement of pipes should be carefully evaluated. Means of avoiding large losses with directly-opposed laterals are discussed in the following.

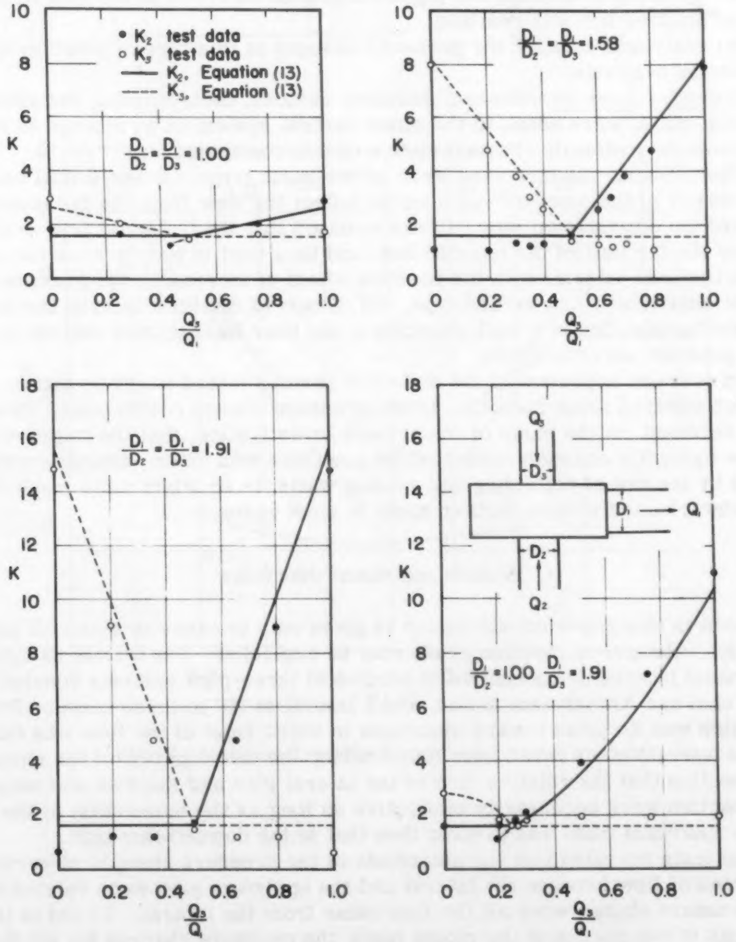

Fig. 8. Results for in-line opposed lateral system.

Improving Opposed-Lateral Flow

Offset Laterals.—Simply offsetting the laterals in plan as indicated in the inset in Fig. 9 proved to be the most effective way of improving the flow characteristics of opposed-lateral flow with rectangular junctions. On the basis of the tests and on consideration of the area over which significant pressures are exerted by a deflected jet, it was concluded that to be most effective the offset of the lateral centerlines should be at least equal to the sum of the lateral diameters.

In Fig. 9 the experimental data on the offset laterals are presented. It will be immediately remarked that the losses were reduced considerably as compared with the directly-opposed laterals. In these tests the pressure change coefficients were always below 2.5 times the outfall velocity head, and generally were below 2.0. Perhaps even more important, the coefficients varied between very narrow limits throughout the practical range of flow divisions. Thus by simply realigning the pipes a material increase in the flow efficiency of the junction box was realized.

No analytical solution for pressure changes at this type of junction is presently available.

Deflector Devices.—Several deflector devices, incorporating deflecting and guiding walls, were added to the offset-lateral system in an attempt to further improve the hydraulic characteristics of this configuration.

The deflector devices used were of two basic types. In one a wall across the corner of the junction was used to deflect the flow from the far lateral toward the downstream pipe. It was realized that the deflected flow would follow the far wall of the junction box, and thus tend to pass across the exit from the near lateral, with the possible effect of increasing the pressure level in the near lateral. A second type, with a curved deflector across the far lateral terminating in a wall extended to the near lateral, was evolved to avoid this possible adverse effect.

An over-all appraisal of the deflector devices tested would be that in general they effected some reduction in the pressure change coefficients. However, it is believed, on the basis of the present investigation, that the improvement in the hydraulic characteristics of the junctions with offset laterals, brought about by the use of deflecting and guiding walls, is so slight as to make their use unwarranted without further study in most cases.

Square and Round Junctions

Since in this paper consideration is given only to cases in which all pipes flow full, changes in pipeline grade may be neglected. The testing of square and round junctions was limited to studies of three-pipe systems consisting of upstream and downstream mains, and a lateral at 90° to these mains. Primary attention was directed toward situations in which most of the flow was carried by the lateral, since it had been found during the investigation of the rectangular junction that the relative size of the lateral pipe and the size and shape of the junction were necessarily ineffective so long as the momentum of the flow in the upstream main was greater than that in the downstream pipe.

The tests revealed that the magnitude of the pressure changes at various divisions of flow between the lateral and the upstream pipe were related to the pressure change when all the flow came from the lateral. To aid in the analysis of the results of the model tests, the pressure changes for all flow

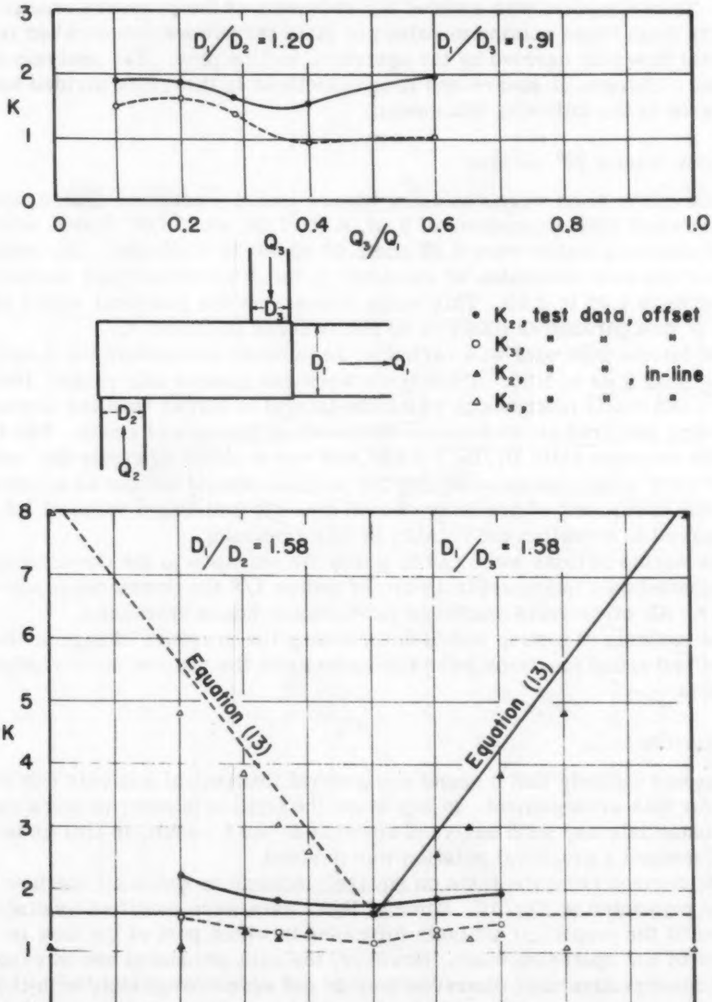

Fig. 9. Results for offset opposed lateral system.

from the lateral were first systematized with respect to pipe size and junction size. Then a method was evolved for reduction of the pressure change coefficients from these maximum values to yield the proper values when less than the total flow was carried by the upstream in-line pipe. The analysis of pressure changes at square and round junctions is therefore divided into these two parts in the following discussion.

All Flow from a 90° Lateral

Most of the tests were run using square model junctions. These square junctions had side dimensions of 6.00, 6.25, 7.00, and 10.00 inches, while the round junctions tested were 6.88 and 9.88 inches in diameter. The ratio of the junction side dimension or diameter to the downstream pipe diameter ranged from 1.05 to 2.10. This range represents the practical extent of the value of this parameter likely to be encountered in practice.

The lateral pipe size was varied so as to cover adequately the range of D_1/D_3 from 0.83 to 1.53. A few tests were run outside this range. Data for square and round junctions at which the lateral is larger than the downstream pipe were required as such cases can occur at changes of grade. The test with the extreme ratio $D_1/D_3 = 0.656$ was run in order to verify the contention that at very small values of D_1/D_3 the junction should behave as a large reservoir, the coefficient of pressure change thus approaching a value of 1.5. This test served to establish the validity of this reasoning.

Two series of tests were run in which the entrance to the downstream pipe was rounded on a quarter-circle arc of radius 1/8 the downstream pipe diameter. All other tests employed flush, sharp-edged entrances.

The methods of testing and of determining the pressure change at the square and round junctions were the same as in the case of the rectangular junctions.

Test Results

It seems unlikely that a sound and general theoretical analysis can be devised for this arrangement. In any event the limited number of tests run did not substantiate any such analyses attempted. As a result, in lieu of an analytical method a graphical solution was devised.

Data derived from the tests on square junctions in which all the flow turned 90° are presented in Fig. 10. Some of these data were modified as discussed later to fit the empirical analysis for cases in which part of the flow is carried by the upstream main. However, the data presented are in general on the conservative side wherever they do not agree completely with the tests.

The curves shown in Fig. 10 are for constant values of the ratio of the square junction side dimension to the downstream pipe diameter. They were faired in to best fit the experimental data, guided by the fact that the pressure change for the lateral must be controlled to a large extent by the momentum of the flow therein. Thus the curves should be approximately parabolic in shape, reflecting the presence in momentum considerations of the square of the pipe-diameter ratio.

To be noted in Fig. 10 is the effect of the junction size. As this is decreased relative to the downstream pipe diameter the pressure loss is also decreased appreciably. This is attributed to the fact that in a very small square junction the wall opposite the lateral is nearly flush with the downstream pipe and thus deflects the flow into this pipe. As the junction is made

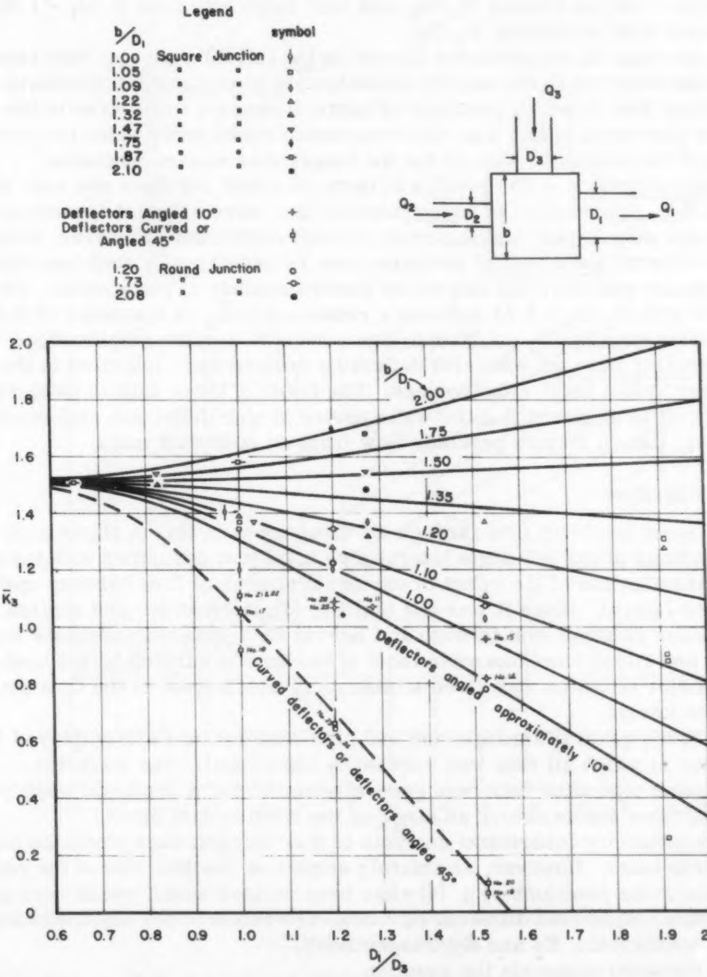

Fig. 10. Lateral coefficients for square and round junctions with all flow from lateral.

larger relatively, the effect of the wall is reduced and in such junctions the lateral flow may substantially pass the entrance to the downstream pipe, thus adversely affecting the pressure change. These contentions are further supported by tests involving deflectors. In moderately large junctions straight walls placed flush with the side of the downstream pipe and opposite the lateral exit, effecting a reduction in junction width on that one side, exhibited significantly beneficial effects.

In addition to the general effect of relative junction size on the pressure change coefficient $\bar{K}_3$, two effects of the size of the lateral relative to the size of the downstream pipe may be noted in Fig. 10. With small junctions, $\bar{K}_3$

decreases with increasing D_1/D_3, and with large junctions ($b/D_1 > 1.50$), $\bar{K}_3$ increases with increasing D_1/D_3.

An increase in the pressure change as the lateral pipe size was reduced was also observed in the earlier investigation of rectangular junctions with combining flow in which junctions of large dimension transverse to the downstream pipe were used. The data from these model tests aided the determination of the curves of Fig. 10 for the larger size square junctions.

Round Junctions.—The results of tests on round junctions are also included in Fig. 10. Apparently the round junction is a more efficient transition than its square counterpart under certain limited conditions. However, definite conclusions on the extent of pressure loss reductions with such junctions are not possible with only the extremely limited number of tests made. Two sets of tests with $D_1/D_3 = 1.53$ indicate a reduction in $\bar{K}_3$ on the order of 0.6. Two other tests with $D_1/D_3 = 1.20$ disclose a reduction in the neighborhood of 0.2.

Deflecting Devices.—Several deflecting devices were installed in these junctions to test their effectiveness. The trend of these data is indicated in Fig. 10. It is apparent that the walls having larger deflection angles are more effective, though they do preclude flow from an upstream main.

Combining Flow

The tests involving flow through all three pipes of this configuration were continuations of the 90°-angle lateral pipe tests just described with the addition of an investigation of the effect of various divisions of flow between upstream main and lateral. Since it was felt that Eq. (4), derived for and applied to the rectangular junction with through and lateral flow, would be adequate for square and round junctions when most of the flow is carried by the upstream main, major attention was given situations in which most of the flow entered from the lateral.

The same general arrangement and pipe-size ratios were employed as in the cases in which all flow was carried by the lateral. The main-to-downstream pipe-size ratio was generally small, but a few tests were run with upstream mains nearly as large as the downstream pipes.

No satisfactory theoretical analysis of flow through such junctions has yet been formulated. However, an entirely empirical modification of the equation for rectangular junctions—Eq. (4)—has been devised which yields very promising results. Somewhat different equations are required for the upstream and lateral coefficients, K_2 and K_3, respectively.

For the upstream main the equation

$$K_2 = \bar{K}_2 \left[1 - \left(\frac{Q_2}{Q_1}\frac{D_1}{D_2}\right)^2\right] \tag{14}$$

is proposed, in which $\bar{K}_2$ is the value of K_2 when all the flow enters the manhole from the lateral. Values of $\bar{K}_2$ are shown in Fig. 11.

For the lateral pipe the equation

$$K_3 = \bar{K}_3 \left[1 - \left(\frac{Q_2}{Q_1}\frac{D_1}{D_2}\right)^{2\,D_1/D_3}\right] \tag{15}$$

is proposed, in which $\bar{K}_3$ is the value of K_3 when all the flow comes from the lateral. Values of $\bar{K}_3$ are shown in Fig. 10. The variable exponent was devised to reflect the fact that at large values of D_1/D_3 the curvature of plots of K_3 *vs.* Q_3/Q_1 is greater than at small values.

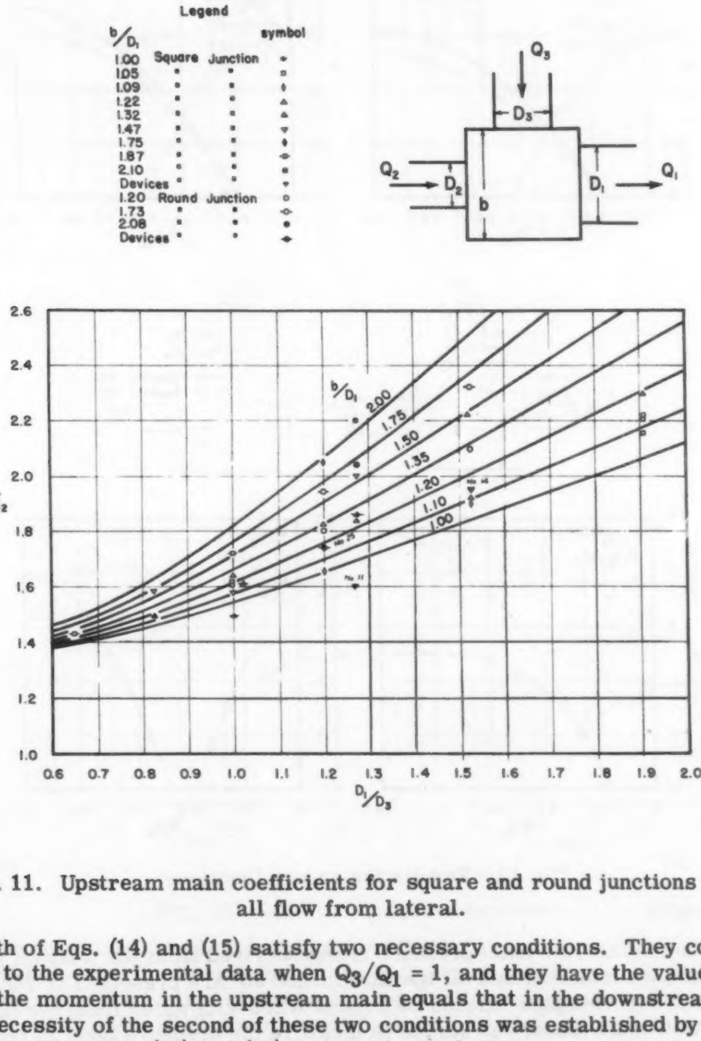

Fig. 11. Upstream main coefficients for square and round junctions with all flow from lateral.

Both of Eqs. (14) and (15) satisfy two necessary conditions. They correspond to the experimental data when $Q_3/Q_1 = 1$, and they have the value zero when the momentum in the upstream main equals that in the downstream main. The necessity of the second of these two conditions was established by the test results. That Eqs. (14) and (15) do indeed satisfy this requirement is evident if it is recognized that the quantity $(\frac{Q_2}{Q_1} \frac{D_1}{D_2})$ is nothing more than the square root of the momentum ratio between the upstream and downstream mains. To obtain values of K_2 and K_3 for lateral discharge rates less than those prevailing at equal upstream and downstream momenta, resort must be had to Eq. (4).

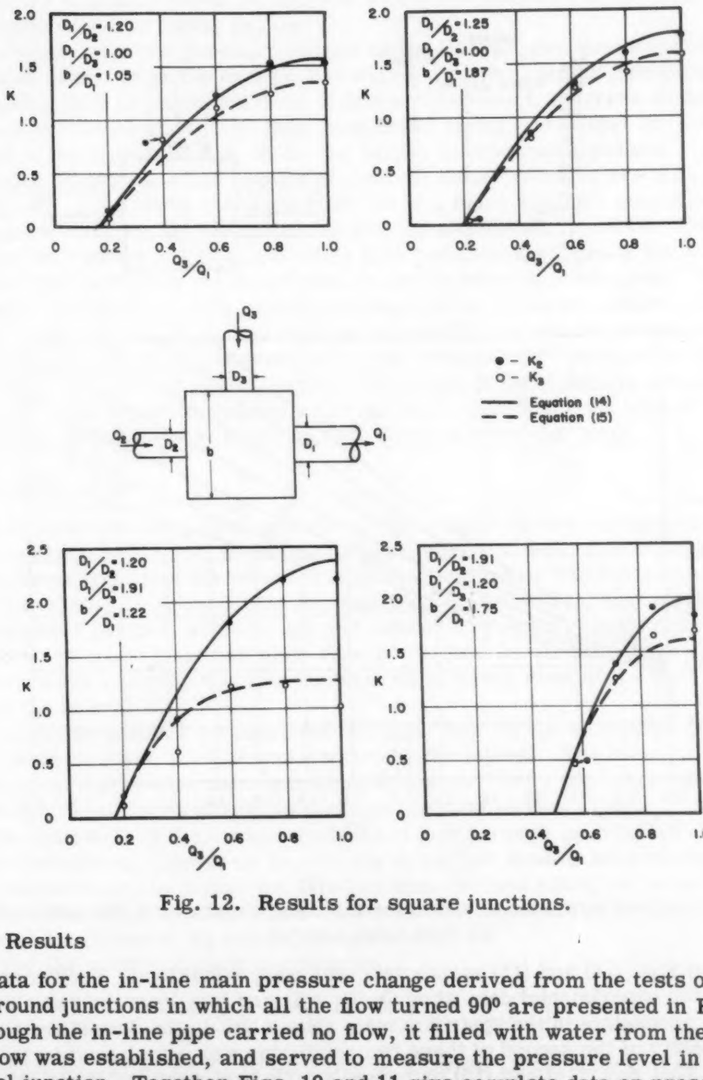

Fig. 12. Results for square junctions.

Test Results

Data for the in-line main pressure change derived from the tests on square and round junctions in which all the flow turned 90° are presented in Fig. 11. Although the in-line pipe carried no flow, it filled with water from the lateral as flow was established, and served to measure the pressure level in the model junction. Together Figs. 10 and 11 give complete data on pressure changes for conditions in which all flow turns 90°.

The curves shown in Fig. 11 are for constant values of the ratio of the junction side dimension or diameter to the downstream pipe diameter, as in Fig. 10. The plotted points for $\bar{K}_2$ at $Q_3/Q_1 = 1$ in Fig. 11 are obtained by plotting K_2 against Q_3/Q_1 and then extending curves through the values for $Q_3/Q_1 < 0.85$ in accordance with Eq. (14) to obtain a corrected value for $\bar{K}_2$ at $Q_3/Q_1 = 1$. The procedure of plotting points and drawing curves in Fig. 11 was similar to that described for the construction of Fig. 10.

Plots of K_2 and K_3 vs. Q_3/Q_1 from the test data revealed that both often reached maximum values with some flow from the in-line pipe rather than at

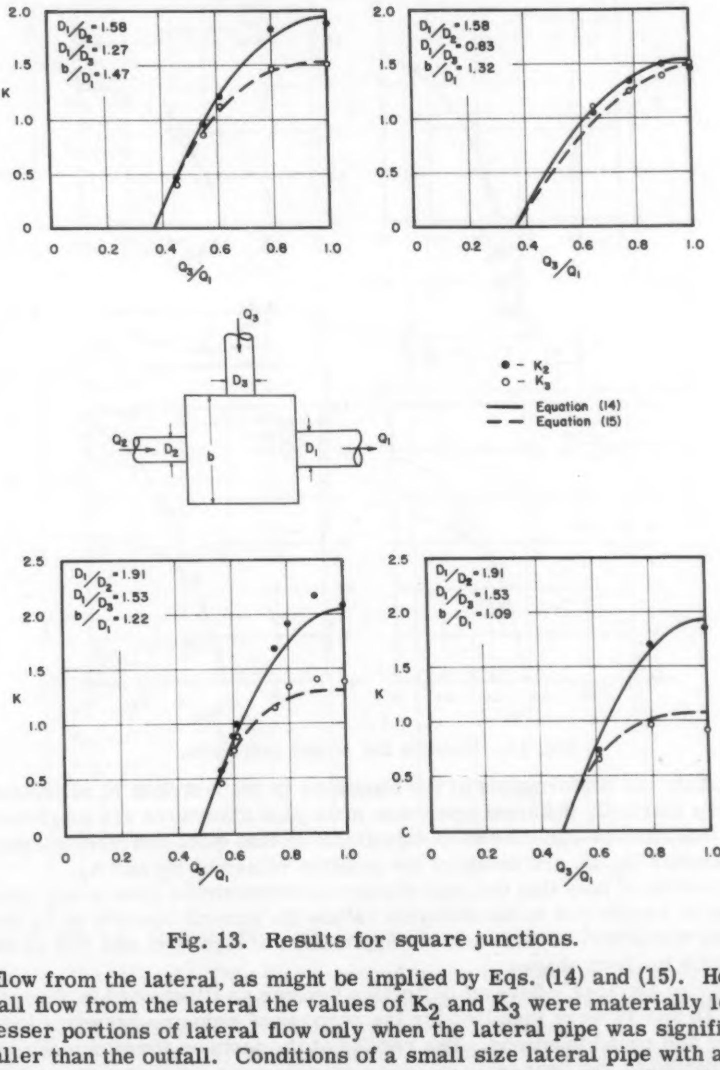

Fig. 13. Results for square junctions.

all flow from the lateral, as might be implied by Eqs. (14) and (15). However, for all flow from the lateral the values of K_2 and K_3 were materially less than at lesser portions of lateral flow only when the lateral pipe was significantly smaller than the outfall. Conditions of a small size lateral pipe with a high proportion of the flow are not common in field practice. Therefore the methods used to determine $\bar{K}_2$ and $\bar{K}_3$ when all flow enters from the lateral are considered to be justified. Any errors in evaluating pressure changes are on the side of overestimation, and become quite small in the practical range of flow divisions in the small size lateral pipes.

Several typical plots of observed values of K_2 and K_3 vs. Q_3/Q_1 are shown in Figs. 12-14. The curves shown represent values given by Eqs. (14) and (15) based on the $\bar{K}_3$ and $\bar{K}_2$ values of Fig. 10 and 11, respectively. The curves in Fig. 14 were obtained through use of the individual values of $\bar{K}_3$ for round junctions as given in Fig. 10 rather than from the curves, which apply strictly only to square junctions.

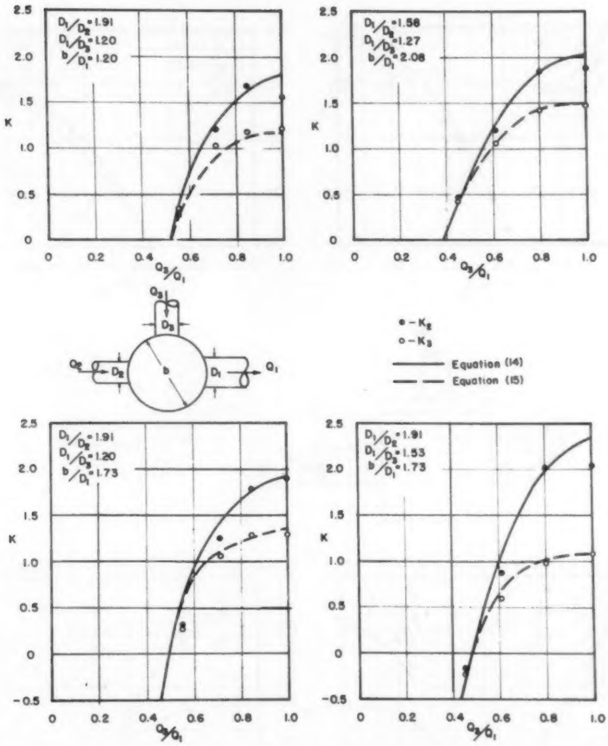

Fig. 14. Results for round junctions.

Obviously the conformance of the equations to the test data is satisfactory, even when markedly different upstream main pipe diameters are employed. This is indicated in Fig. 12 and is significant in that quite different ranges of the parameter Q_3/Q_1 are involved for positive values of K_2 and K_3.

It is worthy of note that the only distinction between the square and round junctions is manifested in the different values (in general lower*) of $\bar{K}_3$ required for the latter. Otherwise, the application of Eqs. (14) and (15) is exactly the same for both shapes.

Deflecting Devices.—Several deflecting devices as illustrated in the insets in Figs. 15 and 16 were installed for the purpose of reducing pressure losses at square and round junctions. The results of the tests on these devices are also presented in the figures.

Several conclusions can immediately be drawn from these graphs and others were based on test results which are not included in Figs. 15 and 16.

Corner deflectors intended to turn the flow from the lateral and reduce its impingement on the through flow, as incorporated in Devices No. 8 and 11 (Fig. 15), were generally ineffective in reducing pressure losses. In fact Device No. 8 had an adverse effect on the lateral pressure loss for large values of Q_3/Q_1 due to the throttling effect at the deflector.

Rounding or shaping of the junction bottom also proved to be ineffective. As evidenced by Device No. 15 (Fig. 16), the pressure loss for the upstream main was markedly increased by the rounding when most of the flow was

*This point is more amply proved by data not included in this paper.

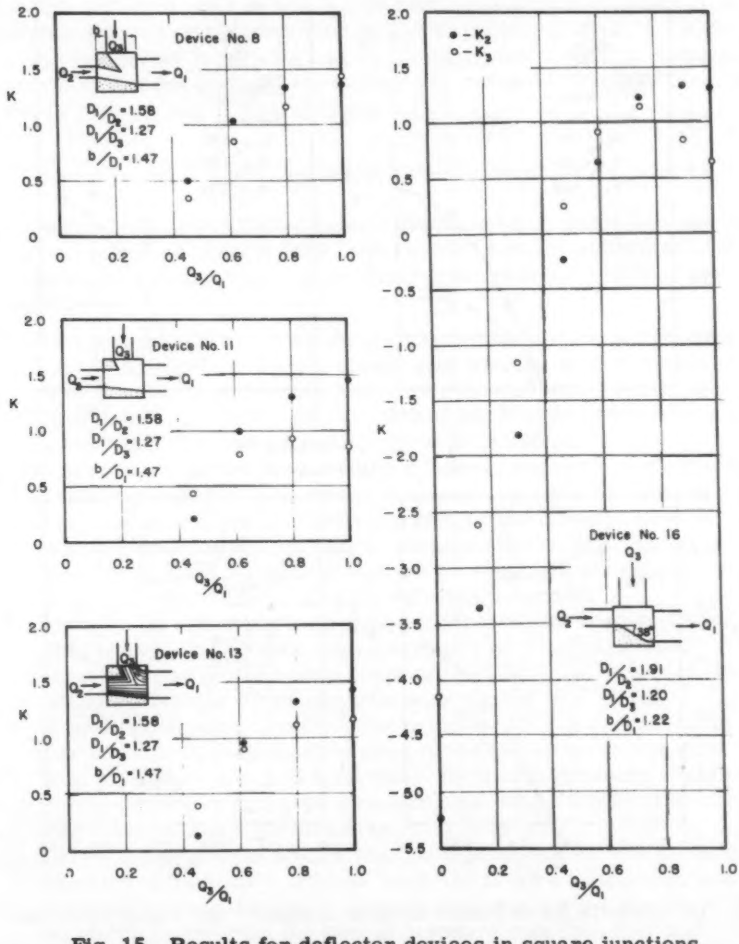

Fig. 15. Results for deflector devices in square junctions.

carried by the lateral. This was ascribed to the tendency of the rounded bottom to deflect the jet from the lateral upward thereby causing the jet to be diffused more thoroughly than it was without the rounding.

Device No. 16 (Fig. 15) exhibited some reduction in the lateral pressure loss when almost all the total flow was carried by the lateral. However, at lower values of Q_3/Q_1 with $V_2 \geq V_3$ the loss was increased over that prevailing for the same pipe sizes without the deflector.

Simple wall-type deflectors, extending from the side of the upstream pipe to the downstream pipe and therefore at an angle of about 10° with the through-pipe centerline, as exemplified by Devices No. 14 and 25, proved to be the most effective of those tested. The curves shown in Fig. 16 are plots of Eqs. (14) and (15) applied to values of $\bar{K}_3$ and $\bar{K}_2$ from Figs. 10 and 11, respectively. Significant reductions in the lateral pressure loss resulted from their use without the adverse effects exhibited by the other devices previously discussed.

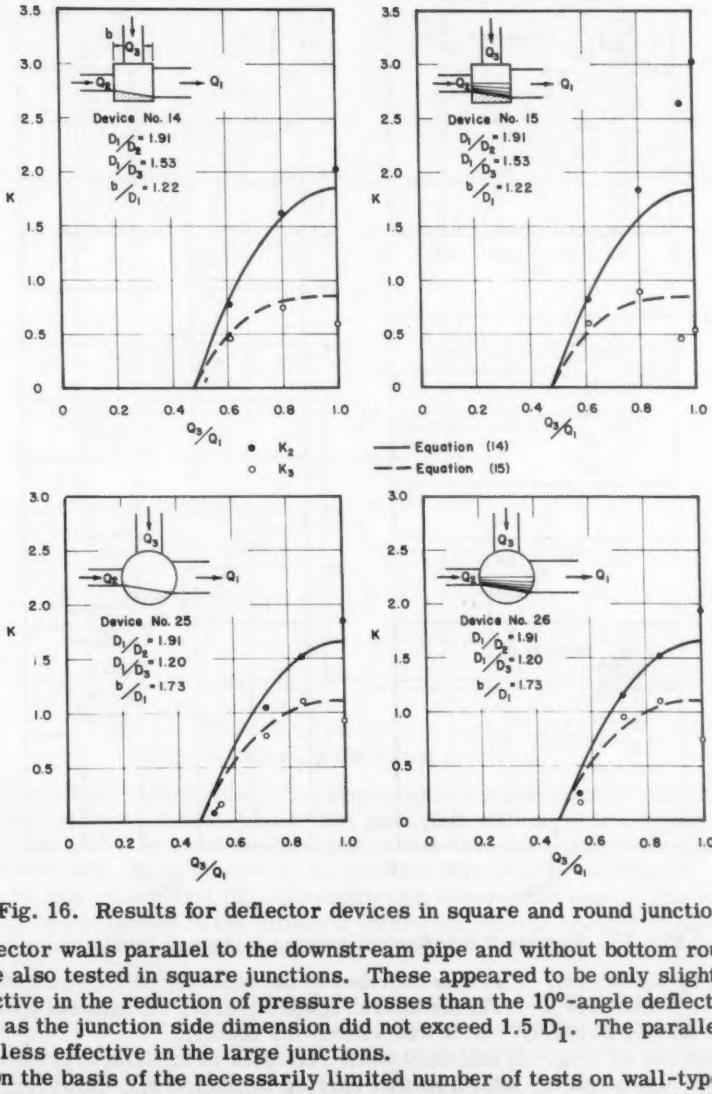

Fig. 16. Results for deflector devices in square and round junctions.

Deflector walls parallel to the downstream pipe and without bottom rounding were also tested in square junctions. These appeared to be only slightly less effective in the reduction of pressure losses than the 10°-angle deflectors, so long as the junction side dimension did not exceed 1.5 D_1. The parallel wall was less effective in the large junctions.

On the basis of the necessarily limited number of tests on wall-type deflectors at an angle to the downstream pipe, the lower two curves in Fig. 10 were added. The tests with parallel wall deflectors, also limited, served to define the curve for $b/D_1 = 1.00$ in Fig. 10. Reduction of K_2 by angled deflector walls is not as great as in the case of K_3. On the basis of the test data available, a curve for $b/D_1 = 1.00$ was added to Fig. 11 for use in defining $\bar{K}_2$ for both zero-angle and 10° deflector walls. Further reduction of K_2 for the angled deflectors does not appear justified. With Figs. 10 and 11 the pressure changes in junctions with the recommended wall-type deflectors may be calculated using Eqs. (14) and (15), just as they may be for junctions without deflecting devices.

The two series of tests in which a rounded downstream entrance was employed indicate an average reduction in $\bar{K}_3$ of approximately 0.25. A reduction of 0.2 may be applied to both $\bar{K}_2$ and $\bar{K}_3$ for purposes of design. Diminished reductions as greater portions of the flow are carried by the upstream main will result through use of Eqs. (14) and (15).

SUMMARY

In addition to the quantitative results discussed previously several qualitative conclusions of interest have been reached and are reiterated below. They apply only to junctions in pipes flowing full with the junction open to the atmosphere.

1. In expanding flow involving an upstream main aligned with a downstream main, shaping of the invert, rounding of the downstream entrance, and other similar modifications heretofore thought to be beneficial were found to be ineffective. On the other hand, projecting entrances to the downstream pipe were found not to be detrimental.
2. In the case of an in-line main and a lateral, rounding of the downstream entrance was found to be effective in reducing pressure losses when large portions of the flow were carried by the lateral. However, reentrant entrances did not exhibit adverse effects in such cases.
3. Directly-opposed laterals should not be employed when they are expected to carry flows having greatly different velocities.
4. Horizontally offsetting such opposed laterals was found to improve measurably the hydraulic characteristics of these junctions.
5. Loss-reducing devices were found to be of little benefit when combined with the offsetting of the laterals as described in 4 above.
6. In square and round junctions the pressure loss is decreased as the junction size is decreased relative to the downstream pipe size.
7. In small square and round juhctions the lateral pressure change decreases as the lateral pipe size is decreased relative to the downstream pipe size. In large junctions the reverse is true.
8. Deflecting devices in square and round junctions were found to be quite effective, particularly in those involving flow only from a lateral. Those deflectors used with three-pipe systems were somewhat more beneficial in reducing the lateral pressure loss than they were for the upstream main.

ACKNOWLEDGMENTS

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Messrs. C. P. Owens and L. R. Burns of the Highway Department and Carl F. Izzard, R. F. Warner, and D. E. Schneible of the Bureau of Public Roads have each contributed valuable advice and assistance. Messrs. J. E. Moulder and R. I. Theron were research engineers during the early phases of the project. The models were constructed with the assistance of Messrs. Roy Thornton and Delbert Morton. The contribution of each of these men is gratefully acknowledged.

Fig. 1. General layout of model system.
Fig. 2. K_2 for in-line system with 90° lateral.
Fig. 3. K_3 for in-line system with 90° lateral.
Fig. 4. Effect of projecting and rounded entrances to downstream pipe.
Fig. 5. Effect of junction width for equal-size pipes.
Fig. 6. Results for two-pipe in-line system.
Fig. 7. Relative mean pressure coefficients for in-line opposed laterals.
Fig. 8. Results for in-line opposed lateral system.
Fig. 9. Results for offset opposed lateral system.
Fig. 10. Lateral coefficients for square and round junctions with all flow from lateral.
Fig. 11. Upstream main coefficients for square and round junctions with all flow from lateral.
Fig. 12. Results for square junctions.
Fig. 13. Results for square junctions.
Fig. 14. Results for round junctions.
Fig. 15. Results for deflector devices in square junctions.
Fig. 16. Results for deflector devices in square and round junctions.

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Proceedings of the American Society of Civil Engineers

FLOODS OF THE FLORIDA EVERGLADES

Edwin W. Eden, Jr.[1]

ABSTRACT

Large areas in central and southern Florida are subject to seasonal flooding, since the natural streams are inadequate to remove excess rainfall during wet seasons. This paper discusses major floods of record in the Everglades and man's efforts to bring that area into productive use through flood and water control.

I. General

The Everglades, in south Florida, are a vast area of low relief, only slightly above mean sea level. Lake Okeechobee, the second largest body of fresh water wholly within the United States, is located at the northerly extremity, as shown on Fig. 1. The Everglades lie in a very shallow trough from 30 to 60 miles wide, extending from the lake to the southern tip of Florida. On the east, a very narrow coastal ridge rising to a maximum of about 30 feet above mean sea level separates the Everglades from a series of coastal lagoons. The largest of the lagoons are Biscayne Bay in the vicinity of Miami and Lake Worth near Palm Beach. On the west, a comparatively wide, sandy area of slightly higher ground separates the Everglades from the Gulf of Mexico and embraces the areas known as the "Devils Garden" and "Big Cypress Swamp." The bottom of the trough slopes from about 14 to 20 feet above mean sea level near Lake Okeechobee to tide level at its southerly end, a distance of about 100 miles.

The surface soil of the Everglades is generally a rich organic peat or muck from 3 to 6 feet deep, with a maximum depth of about 20 feet in isolated spots.

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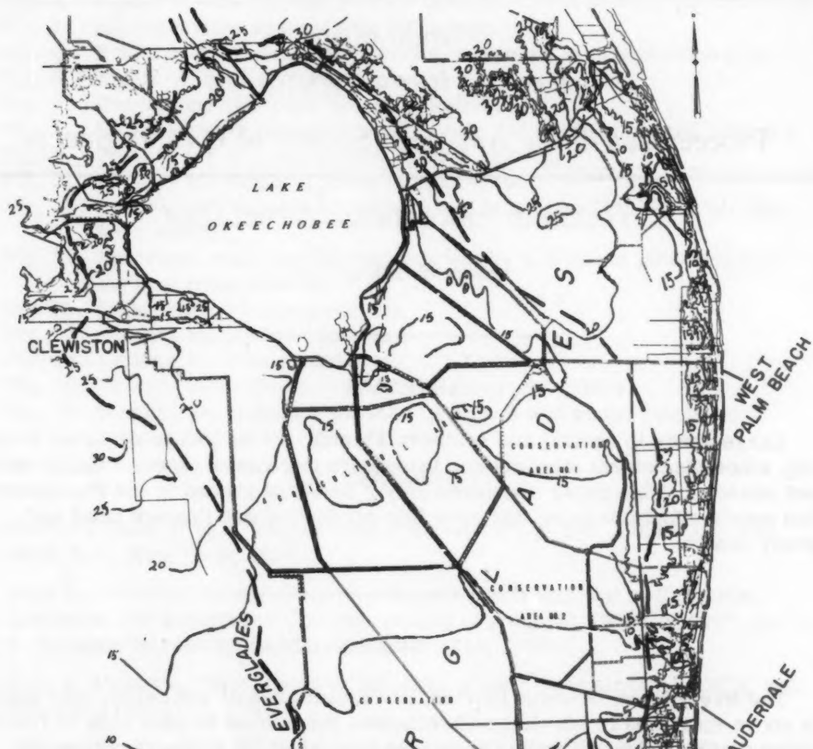

LAKE
OKEECHOBEE
CLEWISTON
WEST PALM BEACH
FT. LAUDERDALE
MIAMI
ATLANTIC OCEAN
EVERGLADES
BOUNDARY OF EVERGLADES
CONSERVATION AREA NO. 1
CONSERVATION AREA NO. 2
CONSERVATION AREA NO. 3
LEGEND
CONTOUR MAP
SOUTHEAST FLORIDA

FIGURE I

The soil has been accumulated gradually over geologic time by the growth and decay of a number of water-loving plants, the most abundant of which are sawgrass and maidencane. Under proper treatment, it can be used to produce winter vegetables, sugarcane, and other crops, or used as improved pasture. However, because of the high organic content, water levels must be regulated closely to minimize subsidence.

The fertility of the soil was recognized many years ago. Just prior to World War I, land developers formed the Everglades Drainage District. That drainage district made the first serious major effort to reclaim the area and develop it for agriculture. Four drainage canals were dug from Lake Okeechobee to the east coast. Each of the canals crossed the upper portion of the Everglades and provided channels through the coastal ridge about 20 miles apart. The drainage afforded by those canals was limited by the extremely flat slopes and the cross section which could be provided with available funds. However, that drainage was sufficient to permit development of lands adjacent to the canals. Since that period, development of the northerly portion of the area has continued at a slow rate.

Most of the area under consideration is included in the Central and Southern Florida Project for Flood Control and Other Purposes (see Sec. IV).

II. Floods of Record

General.—Because of the extremely flat terrain, which is only a few feet above mean sea level, the area has been flooded frequently. Primarily, the flooding is the result of insufficient slopes to convey floodwaters through the extremely thick vegetative cover of the area. It is aggravated during hurricanes when lands adjacent to open-water areas—Lake Okeechobee and the coastal lagoons—are flooded by the storm surge which accompanies such disturbances. Lands bordering Lake Okeechobee have suffered major disasters due to flooding from that source, and coastal areas have experienced serious flooding.

Period Prior to 1924.—Since much of the area under consideration was undeveloped, there are only sparse records of rainfall and water stages for the period prior to 1924. Because of the lack of development, flood damages were very light. Major floods—or hurricanes, which in this area are one of the principal causes of flooding—were reported in 1871, 1880, 1891, 1896, 1898, 1901, 1903, 1904, 1906, 1909, 1913, and 1921.

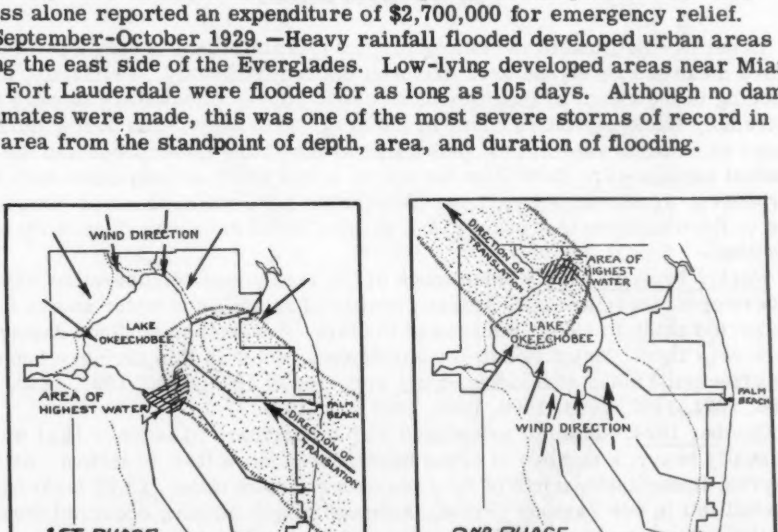
October 1924.—Rainfall associated with a hurricane in October 1924 was unusually heavy; a number of areas experienced more than 20 inches. At New Smyrna, immediately north of the area under consideration, 23.22 inches of rainfall fell in one 24-hour period. Although major flooding occurred throughout the area, damages were low because of the limited development.

September 1926.—One of the most severe hurricanes ever recorded in the United States occurred in September 1926. Before entering the mainland of Florida immediately south of Miami, it generated a storm surge or wind tide of 10 feet in the shallow coastal waters of Biscayne Bay. Damages in the Miami area were concentrated in the bayfront areas and along Miami River where buildings sustained heavy structural damage. Developed coastal areas of Broward and Palm Beach Counties also suffered severe damages. The path of the storm passed over the Everglades immediately south of Lake Okeechobee. The storm created a wind tide in Lake Okeechobee which

overtopped local levees at the southwesterly end of the lake, drowning about 250 people in the vicinity of Moore Haven and causing almost complete destruction of property in the areas affected. Flood-damage estimates were not available, but damages probably reached $2 million or more, on the basis of present values and degree of development at that time.

September 1928.—The storm of September 16-18, 1928, an extremely violent and destructive hurricane, originated in the Cape Verde Islands and entered the mainland of Florida on September 16 at West Palm Beach, crossing directly over Lake Okeechobee (shown on Fig. 2). As the storm struck the lake, winds in the initial phase were from the north, driving the lake waters to heights of more than 12 feet above the stillwater level. About 2,400 persons were drowned and practically all buildings in the vicinity of Belle Glade were destroyed when levees in that area were breached. Four to six hours later, because of the movement of the storm, the winds shifted from north to south. The high tides shifted in response to the change in wind direction and severe high-water conditions were created at the northerly end of the lake near the town of Okeechobee. During the storm, over 2,000,000 acres of land were flooded. The loss of life and property damage in the Lake Okeechobee area make this one of the greatest disasters in the history of this country. No estimates were made of the monetary losses caused by this flood but they were very large in relation to the limited development of that period. The Red Cross alone reported an expenditure of $2,700,000 for emergency relief.

September-October 1929.—Heavy rainfall flooded developed urban areas along the east side of the Everglades. Low-lying developed areas near Miami and Fort Lauderdale were flooded for as long as 105 days. Although no damage estimates were made, this was one of the most severe storms of record in the area from the standpoint of depth, area, and duration of flooding.



DIRECTIONS OF THE HURRICANE, SEPTEMBER 1928

June-September 1930.—The summer of 1930 was unusually wet, with over 20 inches of rainfall reported at a number of stations in June. Conditions were aggravated September 7-11 by a severe storm which deposited an average of about 6 inches of rainfall over the area. Large areas were flooded.

May-June 1936.—During that period, about 8 inches of rainfall over a large portion of the area caused extensive damages.

April 1942.—An unusual rainstorm on April 16 and 17 dumped between 15 and 20 inches of rainfall on the east edge of the Everglades between Miami and West Palm Beach. Damages (1942 price levels) caused by flooding of developed agricultural areas were estimated at $2 million.

September 1945.—A severe hurricane entered Florida near Homestead, at the southerly end of the area. The heavy rainfall associated with the hurricane flooded an estimated 2,000,000 acres and caused damages of about $15 million (1945 price levels). The existence of the Lake Okeechobee levees prevented overflow of the lake with possible subsequent loss of life, although the wind tide experienced was comparable to that witnessed during the 1928 hurricane.

September-October 1947.—The flood of September-October 1947 is the most severe and extensive of record in the area under consideration. It was caused by an abnormally wet summer followed by two hurricanes—the first on September 17-18 and the second on October 9-15. A number of rainfall stations along the easterly edge of the Everglades near the coast reported annual rainfall of over 100 inches, which is 40 inches greater than the normal. As shown on Fig. 3, about 3,000,000 acres of land were flooded for several months. About 100,000 acres of developed farmlands in the agricultural area south of Lake Okeechobee were inundated by the heavy rainfall which was augmented by overland flow from adjacent areas. Emergency flood-protection measures were required to prevent flooding of the towns of Canal Point, Belle Glade, and Pahokee by runoff from high lands lying nearer the east coast. Damages (1947 price levels) in the agricultural area were estimated as follows:

Estimated flood damages (1947 price levels)--1947 flood

Agricultural area south of Lake Okeechobee

Direct damages		
Homes and buildings	$486,000	
Streets and roads	392,000	
Farms and improvements	3,293,000	
Cattle and pasturelands	709,000	
Citrus	13,000	
		$4,893,000
Indirect damages		
Emergency flood measures (including removal of livestock) and loss of business and wages		3,425,000
Total estimated flood losses		8,318,000

The 1947 flood also inundated developed areas along the east side of the Everglades. A good flood-damage survey by the Corps of Engineers estimated the following damages in that area:

Estimated flood damages (1947 price levels)--1947 flood

East coast area

Direct damages		
Homes and buildings	$8,513,000	
Streets and roads	1,185,000	
Farms and improvements	1,417,000	
Farm operation		
Beef cattle	705,000	
Dairy cattle	1,827,000	
Citrus	10,997,000	
		$24,644,000
Indirect damages		
Emergency flood measures and loss of business and wages		17,251,000
Total estimated flood losses		41,895,000

The total losses in the Everglades area from the 1947 flood amounted to more than $50 million.

September-October 1948.—The severe flood losses of 1947 were repeated in September-October 1948 when the area again experienced two hurricanes following a wet summer. The area flooded was approximately the same as that flooded in 1947, although the rainfall was slightly heavier in the southerly portion of the area. A flood-damage survey indicated that damages were slightly less, since sufficient time had not elapsed for complete redevelopment.

August-October 1949.—A hurricane of unusual violence entered the mainland of Florida near West Palm Beach, passing northeast of Lake Okeechobee on August 24. Rainfall associated with the storm was not as severe as that usually experienced. However, rainfall in September was above normal and some flooding occurred. Damage within the Everglades area was small.

September-October 1953.—Accumulation of floodwaters from the almost daily showers over a period of several months caused flooding of undeveloped areas. The completed portions of the Federal flood-control project (see chap. IV) gave partial protection to many areas and prevented about $13,000,000 damages (1953 price levels).

January 21-22, 1957.—Very heavy rainfall was experienced on January 21-22 over the northeasterly portion of the Everglades between Fort Lauderdale and West Palm Beach and extending to Lake Okeechobee. A gage located immediately west of West Palm Beach recorded 21.5 inches of rainfall. The average rainfall over 2,800 square miles was 2 inches or more. Principal damages were to winter vegetable crops nearing harvesting stage. Total damages were estimated at $9,025,000 (1957 price levels). Operation of the pumping facilities provided under the Central and Southern Florida Project

was instrumental in preventing an estimated additional $7,500,000 damages.

December 1957-January 1958.—Unseasonal rains were experienced over developed winter-vegetable areas located between West Palm Beach and Lake Okeechobee. Average rainfall over 2,000 square miles exceeded 4 inches; the maximum observed was 11 inches, at Pumping Station 5A of the Central and Southern Florida Project. Total flood damages were estimated at $1,277,000 (1957 price levels), while operation of the Central and Southern Florida Project prevented additional damages estimated at $8,840,000 (1957 price levels).

III. Flood Problem

Review of the flood history of the Everglades clearly indicates that the major cause of periodic flooding is the extremely flat terrain coupled with the

FLOODED AREAS—1947

FIGURE 3

heavy rainfall experienced in the area. Around the shores of Lake Okeechobee and in the northerly portion of the Everglades, the flooding was tremendously increased by overflow from the lake caused by wind-driven tides prior to the construction of the existing levees. The severity, extent, and duration of flooding have been sufficient to retard development.

In order to obtain some quantitative appraisal of the flood potential, the characteristics of rainfall and wind-tide flooding must be studied individually.

a. Rainfall

The area is subject to wide seasonal variation in rainfall. About two-thirds of the average annual rainfall of from 50 to 60 inches occurs during the wet season—June through October. The mean annual isohyets for the area shown on Fig. 4 have been estimated from data collected by the United States Weather Bureau. Seasonal changes in types of storms are closely associated with variation in rainfall. During summer, which is the normal rainy season, frequent general thunderstorms produce a large portion of the rainfall volume over large areas. This is supplemented by hurricanes, which are the most spectacular producers of heavy precipitation in the area. They are often accompanied by very heavy rainfall near the periphery of the hurricane center, with moderate to heavy rainfall extending a considerable distance and with lighter rainfall over wide areas. Because of their comparative infrequency and the limited area of high precipitation, their overall contribution to water supply is far less than that from other types of rainstorms. However, the record indicates that hurricanes in the past have occurred in a large portion of the flood-damage periods. In any event, the occurrence of severe hurricanes during seasons when non-hurricane rainfall has been unusually large naturally results in excessive flood damages in the area. The tidal flooding around the shores of Lake Okeechobee caused by wind tides is a special flood problem which is analyzed separately.

Analysis of United States Weather Bureau records indicates that there are a number of storms which can be used to estimate the maximum volume of precipitation which can be expected at infrequent intervals. The critical storms were found to be centered somewhat north of the Everglades but from a meteorologic standpoint are considered to be transferable to that area. The maximum rainfall depths for the more critical storms are as follows:

Critical maximum rainfall depths (inches)

Duration*	New Smyrna October 1924		Trenton October 1941		Greenacres City April 1942	
	Station amount	Average over 200 sq. miles	Station amount	Average over 200 sq. miles	Station amount	Average over 200 sq. miles
Hours						
6	13.6	12.2	12.9	9.6	13.1	7.3
24	23.2	20.8	30.0	24.4	15.6	12.1
Days						
2	23.8	21.2	35.0	29.0	18.9	15.3
5	27.9	25.0	35.0	29.2	20.3	16.7

NOTE: *Rainfall amounts for less than 6 hours' duration were not estimated.

MEAN ANNUAL ISOHYETS

FIGURE 4

Since the record indicates that many of the flood-damage periods were the result of prolonged seasonal rainfall, the critical rainfall over longer periods must be considered. The following tabulation indicates the rainfall which has occurred over longer durations within the Everglades area.

Maximum rainfall depth-area-duration data Florida Everglades

Storm period	Duration	Storm center	Maximum rainfall depths (in.)		
			Area		
			10 sq. miles	100 sq. miles	1,000 sq. miles
April 17, 1942	18 hours	Greenacres City	15.4	12.5	7.4
Sept. 4-6, 1933	3 days	Clermont	17.8	17.3	15.7
June 8-17, 1901	10 days	Hypoluxo	19.4	19.2	18.7
Nov. 1-10, 1932	10 days	Canal Point	24.7	20.4	13.9
Oct. 4-23, 1924	20 days	Jupiter	27.0	25.4	22.4
May-June 1936	30 days	Everglades	26.0	25.8	24.9
June-July 1945	30 days	Mountain Lake	28.0	27.6	25.7
July-Oct. 1948	90 days	40-Mile Bend, Tamiami Trail	44.9	44.6	41.9
June-Oct. 1947	5 months	Lake Worth	74.0	68.3	58.2
May-Nov. 1947	7 months	Greenacres City	101.1	92.0	78.5
Feb. 1947-Jan. 1948	12 months	do.	125.0	115.0	98.6

United States Weather Bureau data and standard statistical procedure were used to determine the frequency at which a given depth of rainfall could be expected. The available record at Homestead, Florida—in the southeasterly portion of the Everglades—was used for the study. Rainfall records in the Everglades are practically nonexistent except for recent years; therefore, for long-term periods it was necessary to use records from stations located on the coastal ridge. The frequency of a given depth of rainfall during 1, 2, 10, and 30 days is shown on Fig. 5. It can be seen that about 32 inches of rainfall in 1 month can be expected about once in every 100 years and that 24 inches in 1 month can be expected once every 10 years.

b. Tidal Flooding

In the area adjacent to Lake Okeechobee, the flood problem is complicated by the tidal action of the lake due to wind. The shallow bottom topography of the lake, which is similar to a huge saucer about 40 miles in diameter and about 14.0 feet deep at the center, presents practically the optimum conditions for the development of such tides. Because of those conditions, the behavior of the lake surface has been used to develop basic theoretical relationships between the wind duration and velocity and the tidal action.

In the period 1901 to 1957, there were about 22 hurricanes with winds over 75 miles an hour that crossed the Florida coast. Of those hurricanes, 18 passed within 50 miles of Lake Okeechobee, near enough to exert an influence on the water surface of the lake. Data on each of the storms are given in the following table and the approximate paths are shown on Fig. 6.

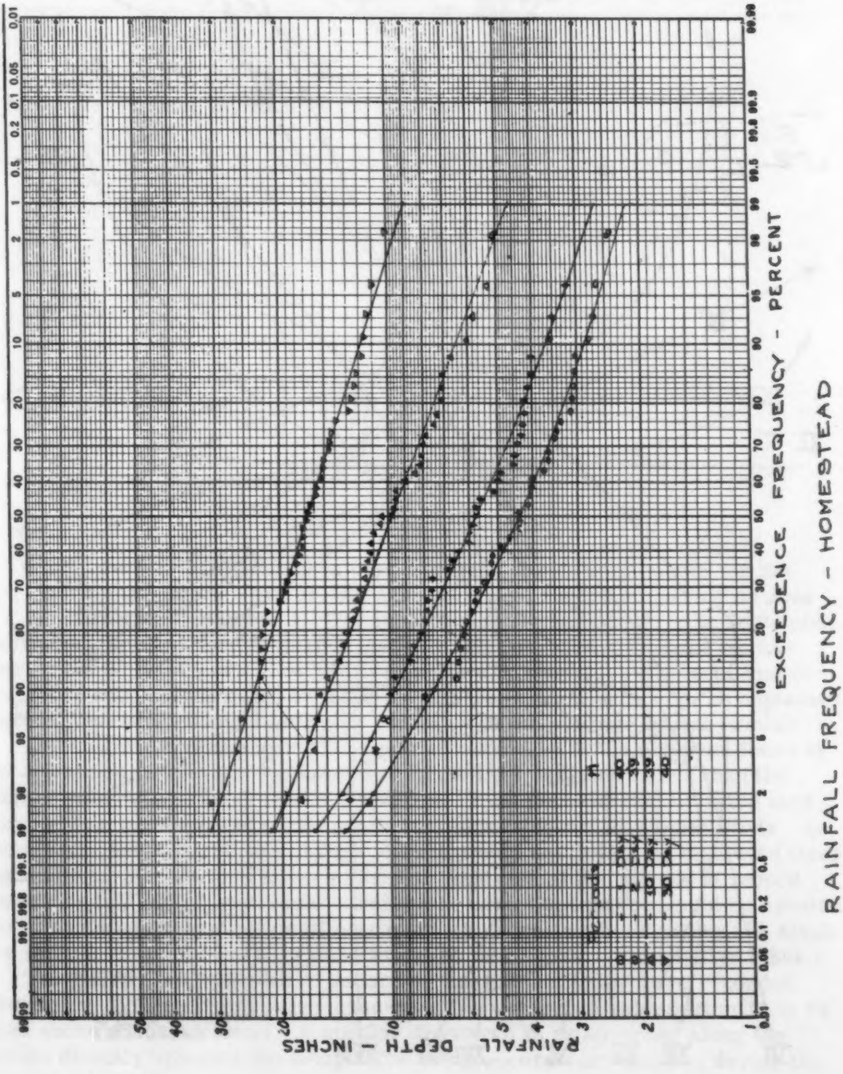

RAINFALL DEPTH - INCHES
EXCEEDENCE FREQUENCY - PERCENT
RAINFALL FREQUENCY - HOMESTEAD

FIGURE 5

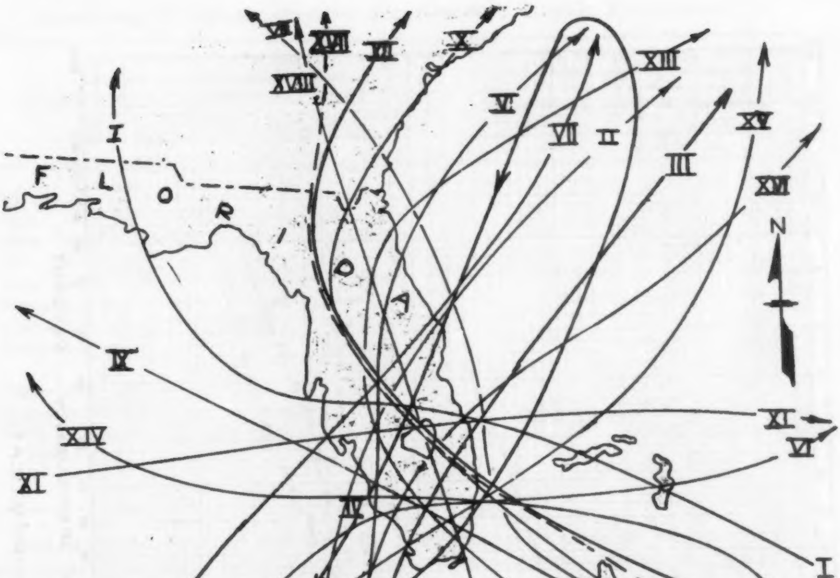

FLORIDA
CUBA
N
Scale in Nautical Miles
HURRICANE PATHS
1801-1953
WITHIN 50 MILES OF
LAKE OKEECHOBEE

FIGURE 6

Hurricanes affecting Lake Okeechobee, 1901-1957

Index No. (1)	Date	Nearest distance to lake (miles)	Minimum central pressure (inches of mercury) (2)	Maximum wind speed (m.p.h.)(2) (3)
I	Sept. 12, 1903	5	29.47	78
II	Nov. 12, 1904	40	29.08	78
III	June 17, 1906	25	29.46	75+
IV	Oct. 20, 1906	10	29.26	75
V	Oct. 18, 1910	30	27.80	84
VI	Oct. 20, 1924	50	28.80	100
VII	Dec. 1, 1925	50	28.90	75+
VIII	July 27, 1926	50	28.80	89
IX	Sept. 18, 1926	50	27.61	110
X	Sept. 16, 1928	0	27.62	117
XI	July 30, 1933	0	29.37	85
XII	Sept. 3, 1933	0	27.98	101
XIII	Sept. 15, 1945	35	28.09	99
XIV	Sept. 17, 1947	35	27.97	103
XV	Oct. 12, 1947	50	29.27	86
XVI	Sept. 22, 1948	0	28.41	85
XVII	Aug. 26, 1949	0	28.17	122
XVIII	Oct. 17, 1950	0	28.20	122

NOTES: (1) Index numbers refer to figure 6.
(2) Extremes recorded.
(3) 5- or 10-minute average, as available.

It can be seen that hurricanes have occurred during each month for the period June through November, the earliest one of record occurring on June 17 and the latest December 1. The largest number has occurred in September and October, with 7 and 5 storms, respectively, occurring in those months during the 57-year period for which records are available. The seasonal occurrence roughly coincides with the normal wet season, although, as explained earlier, hurricanes contribute only a portion of the average annual rainfall.

The magnitude of the wind tide or setup caused by winds has been found to be a function of average wind speed over the lake surface, rather than the highest wind speed found within the storm. Thus the wind tide or setup also varies with the diameter of the circle encompassing the maximum winds. An extremely small hurricane, though violent, would not create as high wind tides as would larger but less intense storms. Analysis of the storms of record indicates that the most critical condition in Lake Okeechobee would be created by a hurricane whose radius of maximum winds was about 20 miles. (A smaller radius, more intense hurricane would be more severe for smaller lakes.)

The maximum wind tides of record on Lake Okeechobee were recorded during the 1928 and 1949 hurricanes when the water was raised from 10 to 12 feet above stillwater level. A similar drawdown is experienced along the shore directly opposite the setup. The setup occurs very quickly; during the August 26, 1949, storm, for example, the water rose on the southeast shore at a rate of over 2 feet an hour and fell at a slightly more rapid rate. Thus the flooding from wind-tide action is sudden, with the water overflowing the lake shores rapidly, if unrestrained by levees, which accounts in part for the

extreme loss of life that was experienced before levees were constructed to confine the lake.

The wide, grassy areas of the lower Everglades have also experienced hurricane winds. However, where the dense vegetation prevents or reduces effective frictional contact between the wind and the water surface, only a minor amount of setup results.

IV. Flood Control Projects

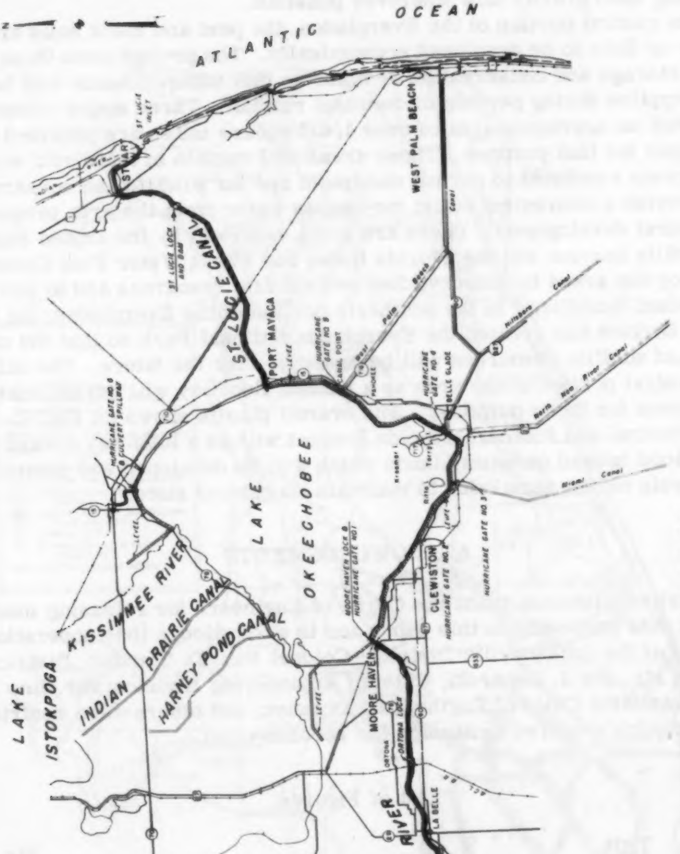
Lake Okeechobee Flood Control Project

After the disastrous floods and hurricanes of 1926 and 1928, Congress authorized the construction of a levee around the south and east shores of the lake to prevent a recurrence of the loss of life and property which occurred during those storms. In addition, the two main outlets from the lake—St. Lucie Canal and Caloosahatchee River—were enlarged to permit more effective control of the lake levels. The levee encircled the lake from Moore Haven to Port Mayaca, at the entrance to St. Lucie Canal. It included all the developed area except the town of Okeechobee. That town is located on slightly higher land and hence is not exposed to the full threat from wind tide. The project (shown on Fig. 7) was authorized in 1930 and constructed during the following decade. Since completion, the project levees have successfully protected adjacent developed lands from inundation by lake waters.

Central and Southern Florida Flood Control Project

Because of their high production potential, the rich lands of the Everglades continued to develop. During World War II, because of high agricultural prices and demand, development was accelerated and vast acreages were improved for the production of winter vegetables and as improved pasture. However, there were other problems in the use of land, such as the excessive subsidence of the soil and actual burning of the soil when recession of the water table permitted it to dry out. In 1935, thousands of acres caught fire and the muck soils above the ground-water table were consumed. After the excessive flood losses of 1947 and 1948, Congress authorized an extension of the Lake Okeechobee project, and the Central and Southern Florida Control Project came into being. The project—now under construction—will provide the facilities required for water control, removal of or protection against floodwaters, supply of water for agricultural areas, control of water levels to prevent excessive loss due to subsidence, and a number of allied purposes to improve utilization of the lands for man and wildlife. The portion of the Everglades which could be considered as usable for long-term agriculture is being provided with the facilities required to decrease the flood hazard. In that area of over 1,100 square miles immediately south of Lake Okeechobee, a major canal network served by pumps was provided since ground slopes are not adequate to remove water by gravity. The pumping stations, which are among the largest low-head stations in the world, can remove three-fourths inch of runoff a day from the drainage area. That removal rate requires a total installed capacity of over 20,000 cubic feet a second.

Lands which can be profitably used for long-term agriculture are also found along the easterly and westerly sides of the Everglades. In those areas,



LAKE OKEECHOBEE
FLOOD CONTROL PROJECT

FIGURE 7

the project will provide the drainage system required for agricultural use, depending upon gravity flow wherever possible.

In the central portion of the Everglades, the peat and muck soils are too shallow or fluid to be developed economically. The project uses those areas for the storage and conservation of water so that adjacent lands will have adequate supplies during periods of deficient rainfall. Three major conservation areas with an aggregate area of over 1,400 square miles are provided within the project for that purpose. Those areas will remain in their wild state, with water levels regulated to permit maximum use for wildlife and conservation. They provide a convenient outlet for excess water from the area proposed for agricultural development. There are plans underway by the United States Fish and Wildlife Service and the Florida Game and Fresh Water Fish Commission to develop the areas to conserve fish and wildlife resources and to provide recreational facilities. In the southerly portion of the Everglades, the National Park Service has created the Everglades National Park so that the natural beauty and wildlife resources will be preserved for the future. The utilization of the central portion of the area as a natural floodway will permit maximum development for those purposes. The overall plan is shown on Fig. 8.

The Central and Southern Florida Project will go a long way toward reducing the flood hazard on usable lands which will be developed and controlling water levels on the remainder to maintain its natural state.

ACKNOWLEDGMENTS

The writer wishes to thank the Corps of Engineers for releasing most of the basic data presented in this paper and to acknowledge the cooperation of personnel of the Jacksonville District—Colonel Paul D. Troxler, District Engineer; Mr. Joe J. Koperski, Chief of Engineering Division; Mr. Leo L. Burnet, Assistant Chief of Engineering Division; and others—who contributed to the research required to develop the records cited.

List of Figures


LEGEND
PROPOSED SPILLWAY
PROPOSED PUMPING STATION
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WEST PALM BEACH
INSERT "A"
HENDRY
BROWARD
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MIAMI
DADE
EVERGLADES NATIONAL PARK
HOMESTEAD
ATLANTIC OCEAN
CENTRAL & SOUTHERN FLORIDA FLOOD CONTROL PROJECT
WEST PALM BEACH CANAL
FLOW
INSERT "A"
SCALE IN FEET

FIGURE 8

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Note.—Paper [illegible] is part of the copyrighted Journal of the Hydraulics Division, Proceedings of the American Society of Civil Engineers, Vol. [illegible], [illegible], [illegible].

NORTHEASTERN FLOODS OF 1955: FLOOD HYDROLOGY[a]

Closure by Elliot F. Childs

ELLIOT F. CHILDS,[1] M. ASCE.—The author appreciates the interest stimulated in flood frequencies. Various methods of analyses and discussions on this controversial subject will continue to plague hydrologists for some time.

Separation of floods into types and causes is apparently gaining in popularity and is being accepted by many. Mr. Beard demonstrates the result of separating flood records into two categories: hurricane and non-hurricane. An additional segregation for the New England area, which would statistically compile the discharges during the spring snowmelt period, is also considered desirable. In fact, the spring runoff peak is the only truly annual event. It is a certainty in New England that the springtime will produce some significant rise in river discharges, which will provide a frequency statistic every year.

The remaining non-hurricane and hurricane floods may not occur every year. However, some years there may be several events in each category and each event should be compiled. This partial duration array is considered essential when the flood-frequency relationship is used for economic analyses of flood control measures.

The author favors the method of segregating floods into types and causes in order to arrive at an envelope or composite frequency curve. However, it is suspected that approximately the same envelope curves have been obtained in Plates 3 and 4 of the author's paper by use of the high skew coefficient of one. Application of such high skew coefficients probably is not statistically sound, but it has the effect of bending the frequency curve to correspond to the one obtained by segregation.

The writer concurs with Mr. Williams that there is considerable merit in developing dimensionless frequency curves in terms of an index flood which may be the annual or 5-year flood. However, a regional or composite curve of frequency plotted versus flood magnitude expressed as a ratio of the mean annual flood, has to be used cautiously in the New England area. Although physical features of basins may be comparable, their location relative to orographic influences on storm rainfall is important. Hydrologic investigations of the New England area indicate that certain watersheds are prone to receiving heavier rainfall amounts than other areas of apparently similar topographic characteristics.

Mr. Foster is quite correct in his reference to the confusion in relating probable maximum precipitation and the maximum possible flood. The Corps of Engineers has adopted the term probable maximum flood, but long usuage

a. Proc. Paper 1663, June, 1958, by Elliot F. Childs.

1. Chf., Hydrology and Hydr. Section, U. S. Army Eng. Div., New England, Waltham, Mass.

of the earlier terminology has created some confusion. The author inadvertently used the former terminology in his paper.

Methods employed by the Corps to revise flood frequency curves since the 1955 flood lead to much higher increases in annual flood losses than noted by Mr. Foster. Plate No. 3 in the author's paper shows revisions in the flood frequencies on the Naugatuck River at Naugatuck, Connecticut. Similar curves have been developed for numerous "damage zones" extending the entire length of the Naugatuck River. A typical damage zone in Waterbury is selected to illustrate the effect of the frequency revisions on the annual flood losses.

Using conventional methods known to flood control economists, curves of (a) stage-damages, (b) stage-discharge, and (c) discharge frequency, are related to obtain damage-frequency curves. The area under the damage-frequency curve, when damage is expressed in dollars and the frequency as percent chance of occurrence in a single year, is the annual flood loss. Keeping all relationships constant, except the flood frequencies, results in changes in annual flood losses shown in Table A. The table is also divided to show the changes within different frequency ranges. It may be noted that the annual flood loss for this particular zone increased 2.7 times merely by the revisions in frequency relationship. This demonstrates the importance of flood frequencies on flood control economics and emphasizes the need for consistent frequency relationships that may be used with confidence in economic analyses.

In 1940 the Corps of Engineers proposed a flood control reservoir at Thomaston in the upper Naugatuck River Bason. At that date the project had a favorable benefit-cost ratio of 1.12, but due to local opposition the dam was

TABLE A

ANNUAL FLOOD LOSSES

NAUGATUCK RIVER AT WATERBURY, CONNECTICUT

Zone 15b

Frequency Ranges in percent chance in a single year	Frequency Curves		
	(a) 1 Records to 1950 Skew = 0.30	2 (a) Records thru 1955 Skew = 0.30	3 (a) Records thru 1955 Skew ¢ 1.00
100 to 5 percent (1 to 20 years)	$218,000	$280,000	$294,000
5 to 1 percent (20 to 100 years)	66,000	116,000	163,000
Less than 1 percent (above 100 years)	39,000	85,000	416,000 (b)
Total annual flood losses	$323,000	$481,000	$873,000

(a) Similar to designation on Plate No. 3 for Naugatuck River at Naugatuck, Conn.

(b) The losses experienced in the August 1955 flood in Zone 15b amounted to $45,000,000.

not constructed. Following the flood of August 1955, which caused $230,000,000 of damage in the Naugatuck River Basin, revisions in the hydrologic analysis, reappraisals of damages, and new cost estimates of the dam, lead to a revised benefit-cost ratio of 5.6. The change in the flood frequency relationship had a major part in this increase. The Thomaston Dam is now under construction.

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WAVE FORCES ON SUBMERGED STRUCTURES[a]

Discussion by John B. Herbich

JOHN B. HERBICH,[1] A.M. ASCE.—This paper is a welcome addition to scarce literature dealing with wave forces on submerged structures and the authors are to be congratulated on obtaining the experimental data and analyzing the problem. Anyone who has been connected with obtaining experimental data in wave channels realizes the difficulty in getting reliable data. Wave reflections from curved walls of the wave tank and from the submerged objects and the presence of traverse wave superimposed on the incident wave, all affect the experimental data.

The writer would like to draw attention to the phenomenon of the transverse waves which apparently were observed in many wave tanks. The presence of transverse waves prevents taking the data on a continuous basis i.e. it is only possible to run the wave generator for short periods of time and delays occur due to waiting between tests for the wave tank to quiet down. The writer had a chance to observe the formation and gradual build-up of transverse waves during a project conducted for the David Taylor Model Basin of the U.S. Department of the Navy.(1) Observations which were made in a 9 ft. wide, 6 ft. deep and 253 ft. long wave tank indicated that the most violent transverse oscillations occurred when the generated incident wave was equal to the width of the wave tank. In this case the transverse wave height was observed to be approximately equal to twice the incident wave height. The transverse waves were also of appreciable height for wave length equal to 3 ft. (or 1/2 width of tank), 12 ft. (twice the width of tank) etc. If generation of waves was allowed to continue for a long time it was observed that the buildup of transverse waves continued until a maximum was reached.

A definite transverse wave pattern was observed on the sloping beach of both the impermeable and permeable wave absorbers installed at the downstream extremity of the wave tank. This pattern which showed the crests and troughs of the transverse wave, depended, of course, on the length of the incident wave generated. When a series of plates were installed vertically on the sloping beach of absorber in the direction parallel to the channel walls, the transverse wave pattern was still present. This would appear to indicate that the absorber was not causing the generation of transverse waves. It does not seem that the transverse waves are set up by the wave generator either, because such transverse waves were observed in wave tanks equipped with various types of generators, i.e. plunger, plate or pendulum type. The

a. Proc. Paper 1833, November, 1958, by E. F. Brater, J. S. McNown and L. D. Stair.

1. Asst. Prof. and Chmn. of Hydr. Div., Lehigh Univ., Bethlehem, Pa.

transverse waves or the transverse disturbance as this phenomenon should perhaps be called occur in different size or width channels and suggest that the problem is a hydrodynamical one. It is hoped that this problem will be investigated both analytically and experimentally as a solution of this problem would be of great value to all the wave tank experimenters.

REFERENCE

1. C. E. Bowers and John B. Herbich "An Experimental Study of Wave Absorbers" St. Anthony Falls Hydraulic Laboratory Project Report No. 54; January 1957.

SNOWMELT RUNOFF[a]

Discussion by Francis T. Schaefer

FRANCIS T. SCHAEFER.[1] —The purpose of the paper is to demonstrate the applicability of an energy budget method of computing snowmelt runoff for a drainage basin for which the usual observational weather data are available. To assess the accuracy of the results obtained by this method, the authors compute daily discharges for a gaging station on the Big Eau Pleine River near Stratford in central Wisconsin, and compare them with the daily discharges published by the Geological Survey. For periods of ice breakup when discharges from the two sources do not agree, the authors conclude that the records of the Geological Survey are in error. In the writer's opinion, some of the authors' conclusions are based upon incorrect assumptions as to the usual behavior of ice. The procedure used by the Survey is not as arbitrary as implied by the authors, but makes use of considerable information not specifically mentioned by them.

The Stratford gage is a continuous recorder in a reinforced concrete house and well 15 feet upstream from a highway bridge. The bridge is a deck-type structure in two spans with a total length of 162 feet between abutments. One pier is located near the center of the channel. A single span railroad bridge is located about half a mile downstream and the river curves in a semicircle between the bridges. The streambed has exposures of bedrock, deposits of sediment, and some scattered boulders. High water and ice result in movements of the bed deposits, causing minor control shifts that are usually effective only at relatively low stages. The river has had a range in discharge during the period of record from no flow to 41,000 cfs with a corresponding range in stage of about 22 feet. Drainage area at the gage is 224 square miles.

Winter flows are usually low, and are commonly less than 10 cfs. Ice may form securely anchored to the banks and to the bed. The water may at times flow through one or more channels restricted by ice on the sides, under a bridge of ice not in contact with the water, or completely enclosed by ice and under pressure greater than atmospheric. Water has also been observed flowing under and over the ice, and with various combinations of the conditions just described. For these reasons the explanation that the increased stage caused by ice is related to the thickness of ice supported by the water is seldom if ever correct. A few examples from field measurements are as follows:

a. Proc. Paper 1834, November, 1958, by J. Harold Zoller and Arno T. Lenz.
1. District Engr., U. S. Geological Survey, Madison, Wis.

Date	Ice thickness, in feet	Indicated backwater, in feet
Jan. 22, 1940	0.4 to 0.7	0.13
Feb. 20, 1940	0.05 to 0.65	1.45
Dec. 13, 1951	0 to 0.3	0.41
Dec. 17, 1958	0.5 to 1.0	0.34

In order to synthesize the discharge figures to agree with those computed from the energy budget data, the authors assumed that the backwater from ice at the breakup would be about equal to that determined from previous measurements and never more than 1.65 feet. To disprove this assumption is difficult for any given station for a specific breakup period on the basis of field measurements. This is because it is usually not possible to make current meter measurements during periods of incipient or heavy ice movement. However, on various occasions throughout the years and at scattered locations the Geological Survey has accumulated measurements during such periods that definitely indicate the backwater variation trend to be expected during breakup periods. Results for two Wisconsin stations are reasonable examples of typical change in backwater. During the winter of 1950 on the Little Wolf River at Royalton current meter measurements of 100 to 300 cfs indicated a backwater effect of 0.18 to 1.24 ft. whereas a current meter measurement obtained during the breakup at a flow of 3,400 cfs indicated a backwater effect of 4.26 feet. During the winter of 1943 on the East Branch Fond du Lac River a current meter measurement of 13.3 cfs on February 8 indicated a backwater effect of 0.50 foot, whereas a later measurement on March 16 at a flow of 1,350 cfs indicated a backwater effect of 5.69 feet. During the 1959 breakup period, observations were obtained on several Wisconsin streams where backwater from ice increased from a few tenths of a foot for flows in the 5 to 50 cfs range to more than 3 feet for flows of 1,000 cfs or more. These conditions occurred on Yellowstone River and Grant River in southwestern Wisconsin. Numerous other examples could be cited to support the general conclusion that as ice breakup and movement commences, the backwater, or ice index, increases appreciably over that defined by prior discharge measurements.

The experienced engineer uses the recorder graph as an index to the variation in backwater during breakup. Changes in slope and erratic fluctuations in stage are parameters used in the extrapolation of values from the control points established by the actual current-meter measurements. The recorder charts, together with temperature graphs, precipitation records, and field notes on stream conditions made by Geological Survey engineers and the local observer furnish the best available index to flows during breakup periods. Jams, of course, affect the backwater conditions radically and erratically. Though it is stated rather indirectly that jams do not occur at Stratford (p. 18), the engineers as well as the local observer have observed jams at this station on various occasions. Specifically, they have been noted in fairly recent years in 1938, 1939, 1942, 1944, 1946, 1949, and 1950. Others, which have gone unobserved, have most certainly occurred. This is evidenced by the rapid fluctuations in gage height recorded during periods of ice movement when it is not logical to attribute such fluctuations to changes in inflow or snowmelt rates.

Figure 1 was prepared from the recorder graph for the March 1945 breakup period as an example of the erratic fluctuations that occur as the ice

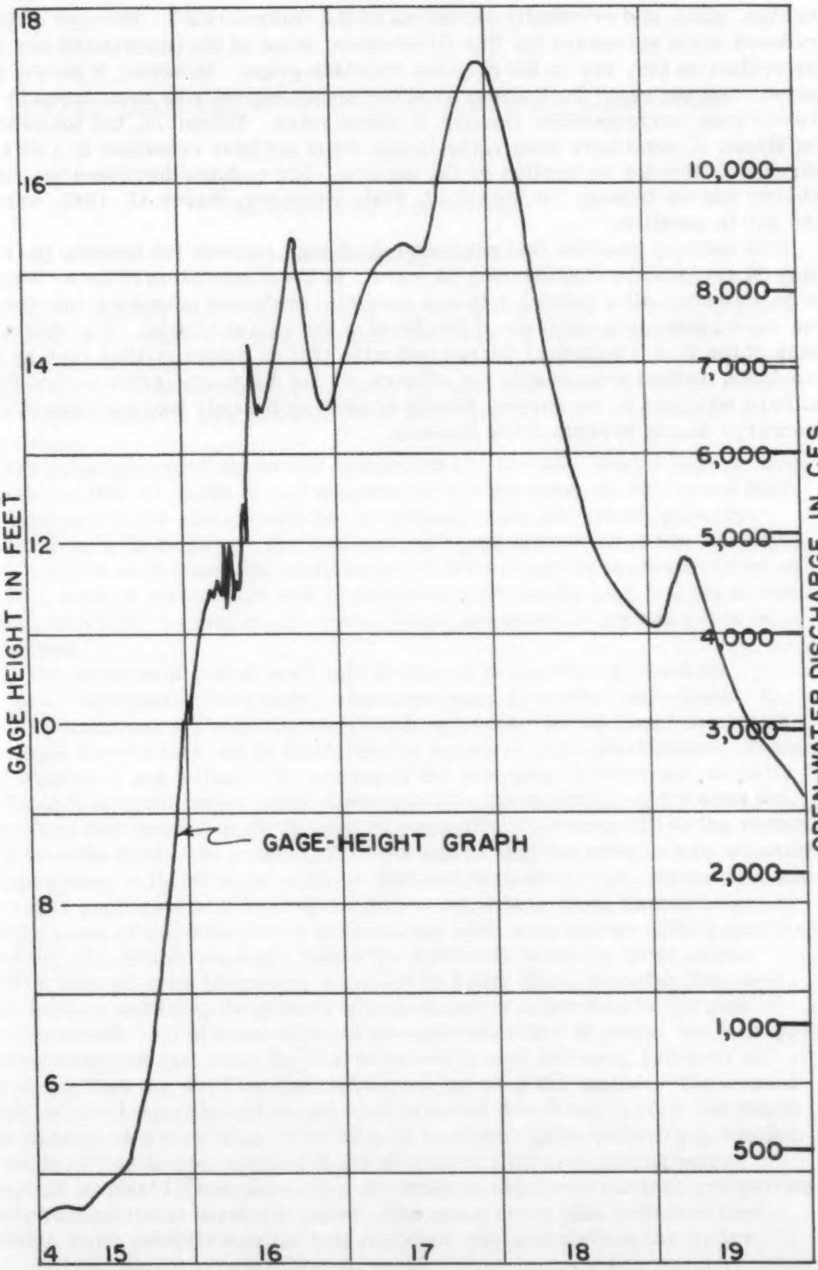

FIGURE 1 GAGE-HEIGHT GRAPH OF ICE BREAKUP PERIOD
BIG EAU PLEINE RIVER NEAR STRATFORD, WIS.

breaks, jams, and eventually moves out of the control reach. Because of the reduced scale necessary for this illustration, some of the fluctuations are not as evident as they are on the original recorder graph. However, it seems apparent that the rapid fluctuations in stage cannot logically be considered to result from corresponding changes in inflow rates. Therefore, the backwater on March 16 must have been variable and could not have remained at 1.65 feet throughout the day as implied by the authors. The authors discussed the discharge and ice indexes for March 17, 1945; obviously, March 16, 1945, was the day in question.

It is entirely possible that published discharge records for breakup periods may on occasion be considerably in error. In the computation of these records, however, all available data are carefully evaluated in order to arrive at the most reasonable estimate of the flows at the gaging station. The experience of the U. S. Geological Survey indicates that no single method such as the ice-index method proposed by the authors, or the discharge-ratio method the authors attribute to the Survey, should be used as the only tool for computing discharge during periods of ice breakup.

METEOROLOGICAL ASPECTS OF STORM SURGE GENERATION[a]

Discussion by J. R. Bowman

J. R. BOWMAN,[1] A.M. ASCE.—Mr. Harris has given us some insight into a number of the meteorological parameters entering into the prediction of surge conditions that are associated with the passage of tropical storms. His presentation of a few equations and generalized statements, however, belies the complexity of the problems involved in dealing with hurricanes and their movements.

The development of electronic computers has brought about a popular misconception that all of the complex mathematical expressions concerned with the behavior of the atmosphere can be automatically and simultaneously solved simply by pushing a few buttons. Although something of this sort may eventually come to pass, the conversion of meteorological forecasts from an art to a work of automatons will require research on the part of a great many human minds for a great many years. Much proposed theory has yet to be developed.

In the absence of tested academic theory, it is customary to set up "models" expressing the general characteristics of weather situations. A model usually has its basis in theory that has been "boiled down" to certain dominant factors that can be evaluated by means of field observations. Table I and Figures 1 and 2 illustrate several of the elements of hurricane models. In Table I, several rather large discrepancies can be observed between the results of the various methods used in computing the radius (R) to the region of maximum winds. In general, it would appear that somewhat more accurate comparisons could be made at some distance outside the eye, where wind and pressure profiles would be less subject to extremely large fluctuations. Although some of the differences in computed radii may appear quite large, they probably fall within tolerable limits for forecasts covering large areas.

The tide patterns (Figure 4) recorded at Sandy Hook, Atlantic City, and The Battery exhibit quite similar characteristics in relation to the path of the hurricane. All of these stations recorded double tide peaks, the first peak having occurred just after the storm center passed offshore, probably at about the time the wind reached maximum speed at the station. The second peak, of about equal height as the first, arrived 6 to 8 hours after the storm had passed, at a time when 35 to 40 knot northerly gales probably prevailed. A study of tide levels reached in the New York City area during extra-tropical storms(17) has shown that the tides at the above stations are particularly responsive to northerly gales. The same study also indicated that Willets Point exhibits similar tide response characteristics; the writer

a. Proc. Paper 1859, December, 1958, by D. Lee Harris.
1. Civ. Engr., Erik Floor & Assocs., Chicago, Ill.

suggests that this factor may account for the seemingly abnormal delay in the arrival of the storm surge at this station.

Much attention has been directed toward the prediction of surge conditions in coastal areas as a hurricane approaches the coast. Seldom do the forecasts deal with potentially severe surge effects on land-locked bodies of water after the storm has passed inland. Actually, there are not many large inland bodies of water that have experienced the passage of hurricanes. To the writer's knowledge, only one such area (Lake Okeechobee) has been subjected to any detailed study; even those studies were not initiated until after the September 1928 disaster in which 1800 lives were lost. Pamlico Sound, in the extreme eastern part of North Carolina, has experienced hurricanes once in every two years, on the average. This sound has a surface area of about 2,000 square miles, and drains an additional area of 25,000 sq. mi. in North Carolina and Virginia. It is bounded on the west and north by the mainland, and on the east and south by natural sand barrier reefs known as the Outer Banks; Cape Hatteras is situated at the juncture of the north-south and east-west banks, and the Weather Bureau's powerful radar station is located about two miles west of the cape. The shoreline of Pamlico Sound is indented by the estuaries of several inland rivers and by a few natural inlets, three of which are considered permanent and provide the sound with its principal outlets to the Atlantic Ocean. The sound is somewhat irregular in shape, but it presents open-water fetches of 20 to 70 miles over depths of 5 to 25 feet.

Winds of full hurricane force have been experienced from every compass direction on the Outer Banks and their water environs, and many instances of dangerously high storm tides have occurred. Two of the permanent inlets are believed to have been cut by hurricane tides emanating from the sound in 1846. In a few cases, storms centered a short distance offshore but traveling parallel to the Banks have produced sound surges that left higher marks than the ocean surges. For example, as hurricane HELENE 1958 approached Cape Hatteras, many persons were evacuated from ocean beach areas to a center situated on higher ground near Pamlico Sound. The expected high tides on the beaches did not materialize, but the minor, yet unexpected, flooding of the evacuation center probably caused some consternation among evacuation officials.

Since the close of World War II, the Outer Banks have developed into popular resort areas. Although they are not yet as intensively populated as either Miami Beach or the Lake Okeechobee area (the latter now safe behind high levees), considerate forecasts of storm surge conditions in Pamlico Sound may help prevent a future disaster of the sort experienced elsewhere in the past.

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EXPERIMENTS ON SELF-AERATED FLOW IN OPEN CHANNEL[a]

Discussion by Michele Viparelli

MICHELE VIPARELLI[1].—This paper makes known the results of long and ingenious research. This research demonstrates that laboratory tests can be useful also in determining air entrainment by water flow.

In 1936, in a very important article on steep channels, Hall L. Standish(1) furnished data collected on big hydraulic structures, but he stated that in his opinion laboratory research on models would have been of small interest only because the laws of similitude are not valid for air entrainment. Now it can be said that the laboratory research in Minnesota, whose results are discussed by the authors of the paper under discussion, and that one in Naples of which results were given by the writer,(2) not only succeeded in realizing some new apparatus of measurements, but they made clear the intimate picture of the phenomenon and permit also general conclusions for prototype design.

In Minnesota recourse has been made to an electric apparatus for measuring air concentration C and to an electric and chemical apparatus for recording instantaneous velocities in a given point of the air and water flow. The local average velocity v was obtained through a statistical analysis of registered data.

In Naples the measured quantities are the dynamic pressure σ on the tip of a Pitot tube; discharge Q_s of the water only, through a vertical strip of unit width; the discharges of air and water syphoned from a point of the current. Therefore, there were used three simple hydraulic apparatus: the common Pitot tube, the open sampler and the closed sampler.

The open sampler, disposed at distance y from the bottom, collects the water discharge Q_s (y) of the current above y. The ratio $q = \frac{\Delta Q_s}{\Delta y}$ gives the water discharge at elevation y, through an elementary unit area, whose normal is in the direction of the flow. The unit water discharge q has the dimension of a velocity and agrees with the average local velocity v in a water flow.

The quantity $\sqrt{2g\sigma}$ was found to depend on the values of $\sqrt{vq}$ according to a linear law. In a current of water only it is $v = q = \sqrt{2g\sigma}$.

In the closed sampler, air and water syphoned from the current became separated; their volumes were measured. The ratio of air volume to the volume of air plus water clearly depends on the air concentration in the point of measurement. But that ratio depends also considerably on the shape of the tip through which air and water are sucked from the current, and on the depression in the closed sampler.

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a. Proc. Paper 1890, December, 1958, by Lorenz G. Anderson.

1. Prof. Ing., Inst. of Hydr. and Hydr. Constr., Naples Univ., Naples, Italy.

After many trials, in the Naples Hydraulic Laboratory for the closed sampler here was adopted a tip with a hole opened towards the direction of the movement, like the tip of a simple Pitot tube. The depression in the sampler was made to assume different values, in order to obtain that one for which the water discharge through the hole of the tip was equal to $q\omega$, in which ω is the area of the hole and q the value of the corresponding unit discharge. The ratio of air volume to the volume of air plus water so collected in the closed sampler was interpreted as the ratio C_t of air discharge to that one of air plus water through the unit area. In effect when water discharge through the hole is equal to $q\omega$, the flow in the point of measurement is not, or almost not, disturbed.

Hence for a flow of air and water it can be written

$$q = \frac{1}{1 + C_t} \, v$$

The air concentrations, C of Authors and C_t just now defined, are absolutely different. Values of C, obtained by means of an electrical apparatus, are depending on air concentration in a plain parallel to the flow; those of C_t are depending on the air distribution in a transversal section of the current. In effect, the writer in Naples tried to measure air concentrations by means of an electric apparatus and he also obtained much larger values than those measured by means of the closed sampler, as above explained.

The Authors and the writer have both admitted that, from the transition surface, particles of water are projected randomly in the overlying region. The measured air concentration C depends on the average number of particles of water that reach or pass through any area parallel to the bottom per unit time, at elevation $y' = y - d_T$ (the Authors define d_T the transition depth). The parallel water concentration is measured by $(1 - C)$, at least for values of C not too large. The Authors have found that $(1 - C)$ is depending on y' according to the cumulative Gaussian probability curve.

The writer admitted that the water particles above the transition conserve a constant average velocity. In this case, the Q_s resulting by the product of average velocity for the number of water particles, which pass through a unit normal strip y', must be distributed according to the cumulative Gaussian probability curve. Experimental data prove that also $Q_s(y')$ is distributed according to the cumulative Gaussian curve.

The conclusions of the authors on water distribution give new proof of the hypothesis made by the writer that the average velocity of water particles above the transition surface assumes a constant value along a normal to the bottom. The other proof was deduced by the writer from the law which Pitot's data depend on unit discharge q.

The authors say that the velocity of flow, measured by means of the chemical-electric apparatus already mentioned, decreases above the transition depth. As in their article the values of measured velocities are not given, the writer thinks that the observed decrease of velocity should be very small. In this case, data referring to upper layers collected in both researches, may be considered thoroughly compatible among them.

The different meanings of C and C_t should merit an investigation; the dependence of the one of the other could illuminate the relation between the average local velocities and the random velocity components of turbulence.

In the present case it can be stated that values of parallel concentration C are scarcely useful in calculations referring to water discharge and mean velocity. They can mislead, because in lower layers the transversal con-

centration C_t is always much smaller than C. In Naples research C_t was found to vary below the transition depth from less than 0,001 for a 11° degree slope to 0,2 - 0,3% for 23° and to 2 - 3% for 45° degree. According to these values of C_t the measured values of $\sqrt{2g\sigma}$, q and v are practically equal in lower layers, at least for slope less than 45° degree (in Naples the larger slope given to the experimental flume was of 45°).

Other results of interest obtained in Naples are the following:

In lower layers the velocity distribution follows the logarithmic law valid for smooth walls if the channel walls are smooth, the law valid for rough walls in other cases.

The depth of transition d_T can be assumed as the height of the portion of current, which contributes to overcome the resistances.

The unit water discharge q at elevation y' above the transition surface is given by the product of the number of water particles, reaching that elevation, for their velocity. This velocity does not vary with y' and is equal to the maximum velocity U of the flow along y'. Consequently, q, like the number of water particles, is distributed according to a Gaussian law. Precisely it is

$$q = U\, e^{-\frac{1}{2}\left(\frac{2g\, y'}{\sigma U^2}\right)^2}$$

where the standard deviation σ was found to depend on the roughness of walls. In effect, σ varies from 0.0020 for smooth walls to 0.0040 for very rough walls.

For $y' = y - d_T = 0$, the total water discharge through a transversal strip in upper layers is

$$Q_s(0) = 0{,}5\sqrt{2\pi}\ \sigma \frac{U^3}{2g}\ .$$

From the foregoing results, the writer was encouraged to draw more general conclusions than the authors did, also because the examination of data gathered by Nitchiporovich(3) on the Gizeldon chute gives him the evidence that results of laboratory research were confirmed by tests on a prototype. That is the reason why the writer summarizes here his conclusions:

In the lower layers, where water flows with air bubbles in suspension, the average velocity can be calculated with same formulae used in calculations referring to currents of pure water, at least for a slope equal or less than 45°. The area, the wetted perimeter and the hydraulic radius are to be computed below the transition surface, because the weight of water particles dispersed in the air does not contribute to overcome the resistance at the walls.

To contain the upper layers, the wall of steep channels must be raised above the transition surface of a height $\frac{c\sigma U^2}{2g}$, where U is the velocity of flow at the transition surface, σ the above said standard deviation, and c a constant of proportionality. It is convenient to give U the value of the maximum velocity in the section, calculated according to the knowledge on uniform flow in open channels.

The constant c cannot be given once for all, but must be assigned in function of the single case. In effect, because water drops are distributed according to the Gaussian law, they can reach increasing heights but in decreasing numbers. So it depends on the inconveniences, which water drops can cause, the opportunity of elevating less or more the wall.

The total water discharge results to be

$$Q = \Omega\ V + b\ 0{,}5\ \sqrt{2\pi}\ \sigma\ \frac{U^3}{2g}$$

being Ω the area of the hydric section below d_T, b the breadth of the channel at transition surface.

These conclusions are acceptable if the flow in the steep channel is regular. But if the inlet or other causes produce transversal waves and irregularities in the flow, they fall in defect.

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TWO METHODS TO COMPUTE WATER SURFACE PROFILES[a]

Discussion by Bernard L. Golding

BERNARD L. GOLDING,[1] A.M. ASCE.—The authors are to be congratulated for their clear, concise presentation of a simple, easy-to-understand variation of the "standard step method" for computing water surface profiles. Their step-by-step solution of the sample problems, as well as their thorough review of the basic hydraulic principles involved, should be of help to the hydraulic engineer in the design office.

"Method A" for computing water surface profiles is quite similar to one developed by the writer in January, 1956 in conjunction with the first major flood plane zoning work which was done under the jurisdiction of the Flood Control Commission of the State of Connecticut as a result of the major floods of 1955 in that region. The late Boris Bakhmeteff in his textbook, HYDRAULICS OF OPEN CHANNELS, first defined the term conveyance, K, equal to $\frac{1.486\ AR^{2/3}}{n}$.

At the start of the flood plain zoning work almost all known methods for computing surface profiles (including the methods described in the Corps of Engineers MANUAL FOR CIVIL WORKS CONSTRUCTION) were tried and backwater curves were computed for various discharges on many of the major rivers in Connecticut. However, the method using conveyance (Method A) proved the fastest and easiest to apply and was adapted and used for all the Connecticut flood studies. There is only one slight difference between the form used by Messrs. Lara and Schroeder as shown on Table 2 of their paper and the method used by this writer. Column 6 of the authors' Table 2 was replaced with two columns, since it was found convenient to define a conveyance K equal to $1.486AR^{2/3}$ and another term $K_d = \frac{K}{n}$. This permitted engineering aids to do the necessary planimetering of area and plotting of conveyance and area curves for each section, tasks which require practically no engineer ing judgment. The engineer then computed the water surface profile at a later date assigning the value of "n" and computing K_d.

This definition of conveyance has also enabled the computation of a single conveyance curve that is applicable to almost all overbank sections. It has been found convenient to plot a single curve of conveyance per unit width for an infinitely wide flood plain (See Figure 3). The conveyance of the overbank is then computed by multiplying the width of the overbank region by the conveyance per unit width for the depth. This method can be used only when the overbank regions are very wide and flat, as they are along many eastern

a. Proc. Paper 1997, April, 1959, by J. Lara and K. Schroeder.

1. Head, Hydrs. Dept., Howard, Needles, Tammen & Bergendoff, New York, N. Y.

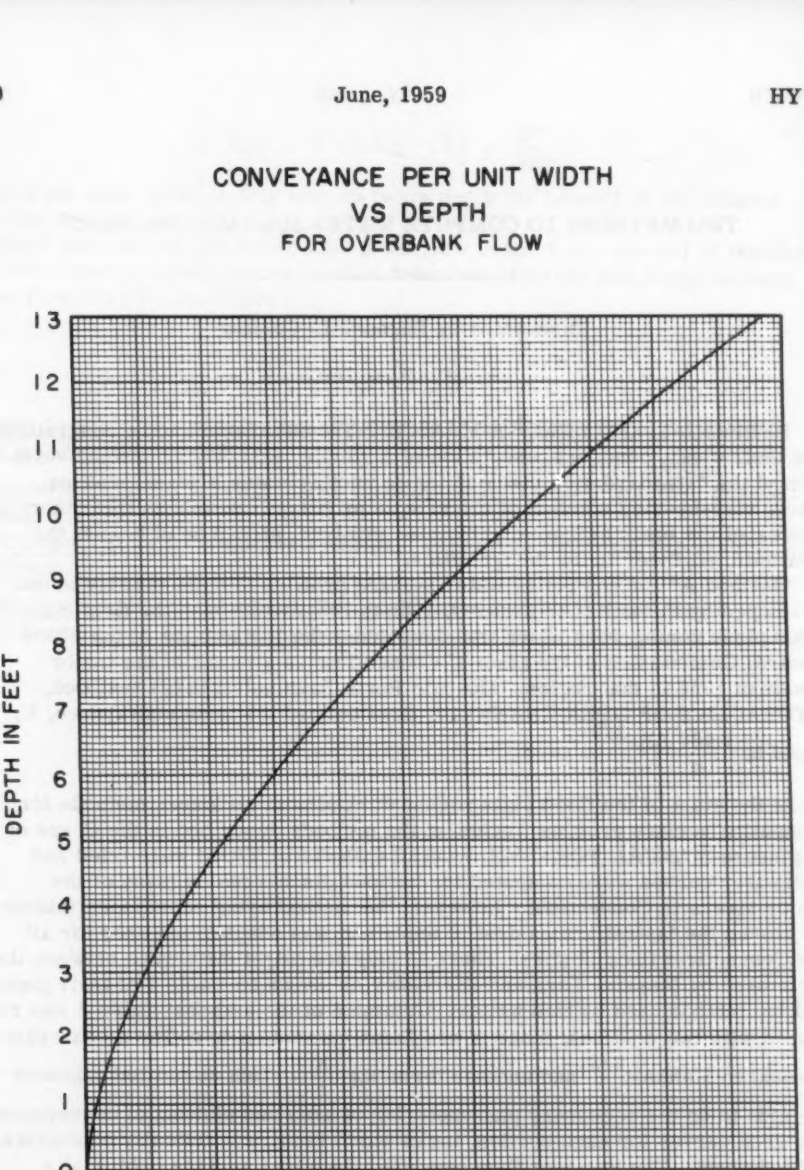
CONVEYANCE PER UNIT WIDTH
VS DEPTH
FOR OVERBANK FLOW
DEPTH IN FEET
0
1
2
3
4
5
6
7
8
9
10
11
12
13
10 20 30 40 50 60 70 80 90 100 110
CONVEYANCE "K" PER UNIT WIDTH = 1.486 D 5/3
HOWARD, NEEDLES, TAMMEN & BERGENDOFF
CONSULTING ENGINEERS
NEW YORK
KANSAS CITY

FIG. 3

rivers.

The writer agrees with the authors that the only practical method for evaluating bend losses is to increase "n," but feels that this may be an unnecessary refinement for bends where the deflection angle is less than 45°. Actually, in attempting to reproduce water surface profiles taken from a survey of high water marks, the writer in the Connecticut work could find no justification for increasing "n" except when the deflection angle was greater than 45°. The difference in travel path lengths between main channel and overbank flow was also considered and tried in several instances (Method B). This refinement also proved unnecessary as it did not affect the computed elevations sufficiently. However, this may not be the case for rivers in the western section of the United States.

The writer would like to call the attention of hydraulic engineers to the new method for computing bridge headwater losses which was recently developed by the Bureau of Public Roads. A paper on this new method titled "Bridge Waterway Design" is available through the Washington, D. C. office of the Bureau. This method allows the computation of bridge head losses where roadway embankments extend a considerable distance into the flood plain. The method also uses the term conveyance as defined by the authors which makes it very easy to apply when using conveyance in the computation of water surface profiles.

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW), divisions. Papers sponsored by the Department of Conditions of Practice are identified by the symbols (DP). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper number are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 1859 is identified as 1859 (HY 7) which indicates that the paper is contained in the seventh issue of the Journal of the Hydraulics Division during 1958.

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JUNE: 1659(AT1), [illegible](AT1), 1660(HY3), 1661(HY3), 1662(HY3), [illegible], [illegible], [illegible], 1666 (PL2), 1667(PL2), 1668(PL2), 1669(AT1), 1670(PL2), 1671(PO3), 1672(PO3), 1673(PL2), 1674(PL2), 1675 (PO3), 1676(PO3), 1677(SA3), 1678(SA3), 1679(SA3), 1680(SA3), [illegible](SA3), 1682(SA3), 1683(PO3), 1684 (HY3), 1685(SA3), 1686(SA3), 1687(PO3), 1688(SA3)[c], 1689(PO3)[c], 1690(HY3)[c], 1691(PL2)[c].

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FEBRUARY: 1933(HY2), 1934(HY2), 1935(HY2), 1936(SM1), 1937(SM1), 1938(ST2), 1939(ST2), 1940(ST2), 1941(ST2), 1942(ST2), 1943(ST2), 1944(ST2), 1945(ST2), 1946(PO1), 1947(PO1), 1948(PO1), 1949(PO1), 1950(HY2)[c], 1951(SM1)[c], 1952(ST2)[c], 1953(PO1)[c], 1954(CO1), 1955(CO1), 1956(CO1), 1957(CO1), 1958(CO1), 1959(CO1).

MARCH: 1960(HY3), 1961(HY3), 1962(HY3), 1963(SA2), 1964(SA2), 1965(SA2), 1966(SA2), 1967(SA2), 1968(SA2), 1969(ST3), 1970(ST3), 1971(ST3), 1972(ST3), 1973(ST3), 1974(ST3), 1975(ST3), 1976(WW1), 1977(WW1), 1978(WW1), 1979(WW1), 1980(WW1), 1981(WW1), 1982(WW1), 1983(WW1), 1984(SA2), 1985(SA2)[c], 1986 (ST3)[c], 1987(WW1)[c], 1988(HY3)[c], 1989(ST3)[c].

APRIL: 1990(EM2), 1991(EM2), 1992(EM2), 1993(HY4), 1994(HY4), 1995(HY4), 1996(HY4), 1997(HY4), 1998 (EM2), 1999(EM2), 2000(EM2), 2001(ST4), 2002(ST4), 2003(ST4), 2004(ST4), 2005(ST4), 2006(PO2), 2007 (HY4)[c], 2008(EM2)[c], 2009(ST4)[c], 2010(PO2)[c], 2011(EM2)[c], 2012(ST4)[c], 2013(PO2)[c].

MAY: 2014(AT2), 2015(AT2), 2016(AT2), 2017(HY5), 2018(HY5), 2019(HY5), 2020(HY5), 2021(HY5), 2022(HY5), 2023(PL2), 2024(PL2), 2025(PL2), 2026(HY5), 2027(HY5), 2028(HY5), 2029(HY5), 2030(SA3), 2031(SA3), 2032(SA3), 2033(SA3), 2034(SA3), 2035(HY5), 2036(SA3), 2037(HY5), 2038(SA3), 2039(PL2)[c], 2040(AT2)[c], 2041(PL2)[c], 2042(HY5)[c], 2043(HY5)[c], 2044(SA3)[c], 2045(HY5)[c], 2046(PL2)[c], 2047(PL2).

JUNE: 2048(CP1), 2049(CP1), 2050(CP1), 2051(CP1), 2052(CP1), 2053(CP1), 2054(CP1), 2055(CP1), 2056 (HY6), 2057(HY6), 2058(HY6), 2059(HY6), 2060(HY6), 2061(SM3), 2062(SM3), 2063(SM3), 2064(SM3), 2065 (ST5), 2066(ST5), 2067(ST5), 2068(ST5), 2069(ST5), 2070(ST5), 2071(ST5), 2072(ST5), 2073(CP1), 2074(HY6)[c], 2075(ST5)[c], 2076(HY6)[c], 2077(SM3)[c], 2078(ST5)[c].

c. Discussion of several papers, grouped by divisions.

HY6

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NO. HY 6
PART 2

Your attention is invited

NEWS OF THE HYDRAULICS DIVISION OF ASCE

AMERICAN SOCIETY OF CIVIL ENGINEERS FOUNDED 1852

JOURNAL OF THE HYDRAULICS DIVISION
PROCEEDINGS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

DIVISION ACTIVITIES
HYDRAULICS DIVISION
Proceedings of the American Society of Civil Engineers

NEWS

June, 1959

PURPOSE OF THE HYDRAULICS DIVISION
(Quoted from the Official Register)

"The advancement and dissemination of knowledge relating to the occurrence of water in nature and its behavior in structures, water courses, and underground.

"In particular the field of the Hydraulics Division shall embrace meteorology and hydrology as they affect the engineer, fluid mechanics in engineering usage, and applied hydraulics as a branch of engineering science which furnishes the basis for hydraulic design and for the practical use of water in the different specialized branches of branches of hydraulic engineering."

EXTRACTS FROM FEBRUARY MEETING OF THE EXECUTIVE COMMITTEE, HYDRAULICS DIVISION

This meeting of the Executive Committee was held on Friday and Saturday, 20 and 21 February 1959, at the Palmer House in Chicago, Illinois. The Committee Chairman, Professor Carl E. Kindsvater, presided. Those in attendance were:

Professor Carl E. Kindsvater, Chairman
Dr. Arthur T. Ippen, Vice-Chairman
Mr. Harold M. Martin
Mr. Maurice L. Dickinson
Mr. Joseph B. Tiffany, Secretary
Mr. Earl F. O'Brien, Contact Member

Among other activities, the Chairman appointed the following contact members to represent the Executive Committee in dealings with the technical and administrative committees:

Note: No. 1959-26 is part of the copyrighted Journal of the Hydraulics Division, Proceedings of the American Society of Civil Engineers, Vol. 85, HY 6, June, 1959.

Committee	Representative
Sedimentation	Mr. Martin
Tidal Hydraulics	Mr. Martin
Publications	Mr. Martin
Hydraulic Structures	Mr. Dickinson
Flood Control	Mr. Dickinson
Research	Dr. Ippen
Hydromechanics	Dr. Ippen
Hydrology	Dr. Ippen
Standards	Professor Kindsvater

The next meeting of the Executive Committee will be held at Denver, Colorado on Tuesday, 30 June 1959, the day preceding the opening of the Ft. Collins Hydraulics Division Conference.

HYDRAULICS CONFERENCE AT FORT COLLINS, COLORADO
July 1 - 3, 1959

The April issue of the Newsletter contains a fairly complete announcement of the eighth national conference of the Hydraulics Division, including a tentative program of the technical sessions. The final technical program and schedules, with the exception of two papers, are as follow:

Wednesday Morning, July 1
Sponsored By The Hydraulic Structures Committee
Presiding: C. E. Kindsvater, Chairman, Executive Committee, and
A. J. Peterka, Member, Hydraulic Structures Committee

THE MANIFOLD STILLING BASIN, Maurice L. Albertson, Director, Research Foundation, Colorado State University, Fort Collins, Colo., and Gene R. Fiala, Highway Design Engineer, U. S. Bureau of Public Roads, Portland, Ore.

THE VORTEX CHAMBER AS AN AUTOMATIC FLOW-CONTROL DEVICE, Richard C. Kolf, Assistant Professor, and Paul B. Zielinski, Assistant Instructor, Marquette University, Milwaukee, Wis.

NEW CONCEPTS IN TUNNEL SPILLWAY DEFLECTOR-BUCKET DESIGN, T. J. Rhone and A. J. Peterka, Engineers, Bureau of Reclamation, Denver, Colo.

Wednesday Afternoon, July 1
Sponsored By The Hydromechanics Committee
Presiding: Arthur T. Ippen, Vice-Chairman, Executive Committee, and
Donald R. F. Harleman, Member, Hydromechanics Committee

SYMPOSIUM ON HYDRAULIC MODELS - PART ONE

A UNIFIED CONCEPT OF DYNAMIC SIMILARITY IN FLUID MODELS, Donald R. F. Harleman, Associate Professor of Hydraulics, Massachusetts Institute of Technology, Cambridge, Mass.

SIGNIFICANCE AND APPLICATION OF FROUDE AND REYNOLDS NUMBERS AS CRITERIA FOR SIMILITUDE, H. K. Liu, Associate Civil Engineer, and M. L. Albertson, Director, Research Foundation, Colorado State University, Fort Collins, Colo.

DYNAMIC SIMILARITY IN WAVE AND TIDAL MODELS, F. Gerritsen, Associate Professor, Coastal Engineering Laboratory, University of Florida, Gainesville, Fla.

SEPARATION AND SCALING OF INERTIA AND VISCOUS FORCES IN PERIODIC FLOWS, Gershon Kulin, Hydraulic Engineer, Fluid Mechanics Section, National Bureau of Standards, Washington, D. C.

Thursday Morning, July 2

Sponsored By The Tidal Hydraulics Committee
Presiding: J. B. Tiffany, Secretary, Executive Committee, and
L. P. Disney, Chairman, Tidal Hydraulics Committee

SYMPOSIUM ON INSTRUMENTATION

NEW INSTRUMENTS DEVELOPED BY THE U.S.G.S. FOR THE MEASUREMENT OF TIDAL FLOW, E. G. Barron, Hydraulic Engineer, Research Section, Surface Water Branch, U. S. Geological Survey, Columbus, Ohio.

RECENT DEVELOPMENTS BY THE COAST AND GEODETIC SURVEY OF EQUIPMENT FOR THE MEASUREMENT AND REMOTE RECORDING OF TIDES AND CURRENTS, A. J. Goodheart, Division of Tides and Currents, U. S. Coast and Geodetic Survey, Washington, D. C.

A PRESSURE-TYPE TIDE AND WAVE RECORDER, E. H. Bowler, Instruments and Control Systems Laboratory, National Research Council of Canada, Ottawa, Canada.

SPECIAL DEVELOPMENTS IN OCEANOGRAPHIC INSTRUMENTS, J. M. Snodgrass, Head, Special Developments Division, Scripps Institution of Oceanography, La Jolla, Calif.

Thursday Afternoon, July 2

Sponsored By The Sedimentation Committee
Presiding: Harold M. Martin, Member, Executive Committee, and
E. J. Carlson, Member, Sedimentation Committee

SEDIMENTATION AND CONSERVATION ON WESTERN ARID LANDS, H. V. Peterson, Area Chief, General Hydrology Branch, U. S. Geological Survey, Denver, Colo.

THE USE OF ULTRASONICS IN THE MEASUREMENT OF SUSPENDED SEDIMENT SIZE DISTRIBUTION AND CONCENTRATION, Gordon H. Flammer, Assistant Professor, Utah State University, Logan, Utah. (Third paper by Simons and Richardson, Fort Collins.)

Friday Morning, July 3

Sponsored By The Flood-Control Committee
Presiding: Maurice L. Dickinson, Member, Executive Committee, and
Arno Lenz, Member, Flood Control Committee.

FLOOD CONTROL PROBLEMS IN THE SOUTH PLATTE RIVER BASIN, COLORADO, Stanley A. Miller, Supervisory Water Conservation Project Engineer, Colorado Water Conservation Board, Denver, Colorado.

CHANGES IN URBAN OCCUPANCE OF FLOOD PLAINS IN THE UNITED STATES, Gilbert F. White, Chairman, Department of Geography, University of Chicago, Chicago, Ill.

A NEW APPROACH TO LOCAL FLOOD PROBLEMS, Gen. Herbert D. Vogel, Chairman of the Board, Tennessee Valley Authority, Knoxville, Tenn.

A SUGGESTED OUTLINE OF LEGISLATION TO ACCOMPLISH FLOOD PLAIN ZONING, author to be announced.

Friday Afternoon, July 3

Sponsored By The Hydrology Committee

Presiding: A. R. Chamberlain, Co-Chairman, Fort Collins Hydraulics Conference, and V. A. Koelzer, Member, Hydrology Committee

SYMPOSIUM OF THE USE OF ELECTRONIC COMPUTERS IN HYDROLOGY APPLICATION OF ELECTRONIC COMPUTERS TO THE SOLUTION OF HYDROLOGY PROBLEMS IN THE BUREAU OF RECLAMATION, Francis E. Swain and H. S. Reisbol, Bureau of Reclamation, Denver, Colorado.

THE ROLE OF ELECTRONIC COMPUTERS IN HYDROLOGIC STUDIES OF THE TVA, W. M. Snyder, Tennessee Valley Authority, Knoxville, Tenn.

COLUMBUS RIVER SYSTEM POWER ANALYSIS BY USE OF DIGITAL COMPUTER, David J. Lewis and Loren A. Shoemaker, Corps of Engineers, U. S. Army North Pacific Division, Portland, Ore.

THE USE OF HIGH-SPEED COMPUTING EQUIPMENT IN WATER RESOURCES INVESTIGATIONS OF THE U. S. GEOLOGICAL SURVEY, G. E. Harbeck, Jr., and W. L. Isherwood, U. S. Geological Survey, Denver, Colorado.

PAPERS ON CHANNEL ROUGHNESS SOUGHT BY HYDROMECHANICS COMMITTEE

The Hydromechanics Committee, through the Task Force on Friction Factors in Open Channels, is in the process of preparing a study of the problem of frictional flow in open channels. In this connection the Task Force is sponsoring the preparation of papers dealing with this subject, and containing ideas or data that may be useful.

Papers may be submitted through regular channels, for publication or presentation at appropriate meetings, or transmitted through the Task Force. Authorship will of course be fully recognized. Dates for publication or presentation will be arranged following receipt. Both ideas and data are needed. Papers giving results of field or laboratory measurements of roughness on fixed and moveable bed channels are desired. Also, measurements for channels of unusual form. To be useful to Task Force, each prepared paper (but not necessarily the oral presentation) should present not only the roughness coefficient in the author's formula, such as Manning's "n," but also all available related data such as length, slope, mean velocity, cross section characteristics, and bed material characteristics. Conversion by the author of his coefficient of roughness to values of "n" and to Chezy's "C" or the Darcy-Weisbach "f" will also be helpful.

Prospective authors are urged to contact Mr. Julian Hinds, P. O. Box 871, Santa Paula, California, who is Chairman of the Task Force.

TIDAL HYDRAULICS COMMITTEE ACTS

The continuing prosperity may be threatened by the industrial, urban and agricultural enterprises bordering many of the tidal estuaries of these United States. Upstream consumptive-use development with its depletion of fresh water inflows to estuaries is steadily permitting farther and farther intrusion of sea water which, in turn, is substantially deteriorating the riparian water supplies to degrees of uneconomical or unpotable usefulness. Disposal of liquid and solid pollutants and contaminants into estuaries and along shores, which has long been dependent upon dilution by inflowing upstream fresh water, must now and in the future depend far more upon the mechanical and chemical characteristics of the estuarial or littoral tidal prisms.

These are some of the knotty facets facing the Committee on Tidal Hydraulics in its task of evaluating and stating the problem of relation and effect of tidal hydraulic phenomena upon the vast array of enterprises which are environmental and riparian to estuaries and shores. During its meeting in Denver, Colorado, on March 19th the committee gave much consideration to activities concerned with the influences of tidal regimen upon local water quality, salinity control, sanitation, sedimentation, and general local usabilities of the riparian estuarial and littoral waters.

The committee is sponsoring technical sessions in Reno in June 1960, and in Seattle in August 1960, and possibly in Boston in 1960, which will be pertinent to this and similar tidal hydraulics problems.

The Committee on Tidal Hydraulics, in the Hydraulics Division, is comprised of L. P. Disney, Chairman, U.S.C. & G.S., Washington, D. C.; E. P. Fortson, Past Chairman, Waterways Experiment Station, Vicksburg, Mississippi; H. G. Dewey, Jr., San Francisco District, U.S.E.D.; Irvin M. Ingerson, California State Department of Water Resources; and Raymond Boucher, Ecole Polytechnique, Montreal, Canada.

FOR YOUR CALENDAR
ASCE Meetings

July 1-3, 1959	Hydraulics Division, Fort Collins Conference
October 19-23, 1959	ASCE, Washington, D.C. Convention
March 7-11, 1960	ASCE, New Orleans Convention
June 20-24, 1960	ASCE, Reno Convention
October 10-14, 1960	ASCE, Boston Convention
April 10-14, 1961	ASCE, Phoenix Convention
October 16-20, 1961	ASCE, New York Convention
February 1962	ASCE, Houston Convention
May 1962	ASCE, Omaha Convention
October 15-19, 1962	ASCE, Detroit Convention

Non-ASCE Meetings

June 15-19, 1959	American Society for Engineering Education, Pittsburgh, Pennsylvania
June 17-20, 1959	National Society of Professional Engineers, New York City
June 18-20, 1959	ASME Applied Mechanics Conference, Virginia Polytechnical Institute, Blacksburg
August 24-29, 1959	International Association for Hydraulic Research, Montreal, Canada
September 9-11, 1959	Midwestern Conference on Fluid and Solid Mechanics, University of Texas, Austin

Deadline dates for Newsletter contributions: August issue - June 20.
October issue - August 20.

USE OF THE HYDRAULICS DIVISION NEWSLETTER

You are urged to continue use of the Division Newsletter for announcements, inquiries, personnel news, committee reports, surveys and other items of interest to Division members. A short note covering the highlights of committee meetings (including task forces) is particularly requested. Suggestions for improvement of the Newsletter will be appreciated.

GUY L. ARBUTHNOT, JR.
P. O. Bos 631
Vicksburg, Mississippi
Newsletter Editor

P.S. Please fill out the form on the inside back cover of the April issue and return it to Mr. Ball.